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American Railway Engineering Association

Bulletin 719

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Front Cover Photo - Union Pacific freight north of Lynndyl, Utah

Rear Cover Photo - Los Angeles bound Union Pacific train leaves summit at Cima, California heading down 2.2% grade.

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Photo 1

Union Pacific's Salt Lake City - Los Angeles Line

For the pictorial cover feature in this issue, photographs of the Salt Lake City - Los Angeles line of the Union Pacific from Smelter, Utah to Barstow, California were chosen. This single-track line is characterized by long stretches of running in wide desert valleys between mountain ridges, and is also notable for dramatic canyon scenery through Caliente Canyon and Meadow Valley Wash in southern Nevada. It has an 18 mile-long 2.2% grade eastbound between Kelso and the summit at Cima, California, which is about 75 miles southwest of Las Vegas. The line has had some alignment changes constructed in recent years (See page 163, AREA Bulletin 692 (Volume 84), May 1983). The 5-mile long project at Crestline, Nevada, shortened the line's distance by 0.87 miles and reduced maximum curvature from 10°02′ to 1°25′. It includes a test section of concrete ties, which appear to be performing well. Other tests are being conducted between this point and Caliente, an area of the line which has many sections of sharp curvature and heavy grades up to about 2%. On this 36 mile section, 8.2 miles are limited to 25 mph or less for both passenger and freight trains, including 4.3 miles restricted to 20 mph. (Speeds over much of the rest of the line are 79 mph for passenger and 70 mph for freight). Photo 1 (above) shows a Los Angeles-bound train crossing a desert geographically north of Lynndyl, Utah. Photo 3 (below) shows a Utah-bound freight in the Meadow Valley Wash geographically south of Caliente, Nevada. Photo 2 (right) shows the test section of concrete ties on the Crestline, Nevada line change. The front cover shows the same train as in photo 1 about a mile further along the line going through the sagebrush and desert grasses that characterize vegetation in this area. The back cover photo shows a Los Angeles bound freight descending the 2.2% grade from Cima, California, amid a landscape dominated by Joshua plants.

Photo 2

Photo 3

Excerpts From Annual Report of A.R.E.A. Committee 1 - Roadway and Ballast

W.S. Stokely, Chairman

Brief status of each Subcommittee Assignment:

Subcommittee 1. Roadbed

Assignment D1-1-86, a report on the justification of the use of under cutters is in its second draft, and expected to be edited to a final draft at the coming Fall meeting.

Assignment D1-2-88, is to update the Manual text on construction of roadbeds. This is a new assignment, and work has not begun account activity on the other assignments. Progress is expected during calendar year 1989 with completion during 1990.

Assignment D1-3-88, is on railroad applications of mechanically stabilized walls. Work on this matter has progressed to a point where a second draft will be available during the coming Fall meeting. Completion is expected in 1989.

Subcommittee 2. Ballast

Assignment C2-1-86, deals with reporting on result of AAR and other ballast research programs. Considerable effort has been expended on this assignment in cooperation with AAR in the design of the FAST/HAL Ballast Tests. This assignment is, of course, on-going to the extent that ballast tests are in progress.

Assignment D2-1-88, dealing with tamping procedures and their relation to the ballast, is new and in its formative stages. A substantial list of resources have been made available to the Subcommittee. We have no time frame for completion of this assignment at present.

Subcommittee 3. Natural Waterways

Assignment D3-1-86, deals with scour prediction and protection. This work has been presented. The second assignment, D3-2-88, on the topic of protection of roadbed and bridges from washout is in the formative stages. Continuation of these assignments is recommended.

Subcommittee 4. Culverts

Assignment D4-1-88 is on the matter of updating the Manual text on installation of pipe culverts, to include jacked installations. This assignment is in its first development stages. Completion is expected in 1990.

Subcommittee 5. Pipelines

Assignment D5-1-74, deals with casing pipes larger than 42″ in diameter. Assignment D5-2-81, deals with plastic carrier pipes. The Board is to note the age and nature of these two assignments. Both are old, and both are highly technical. The Subcommittee Chairman, in his first term, has asked for technical assistance, and Headquarters has responded. Long awaited progress is not anticipated. Both assignments are important to the railroad industry, and the effort to update the Manual must continue.

Subcommittee 6. Fences

Assignment D6-1-87, dealing with security fences, has been reported on out of Subcommittee. The Committee Chairman has returned the work with technical and graphics quality comments. The assignment is to continue to a completion in 1989.

Assignment, D6-1-87, dealing with snow fences is under study and is to continue with completion in 1990.

Subcommittee 8. Tunnels

Assignment D8-1-87 addresses updating of the Manual materials, and D8-2-87, is to report on the Roger's Pass Project. The two assignments go hand-in-hand, and are to continue to a completion in 1989 for the report work, and 1990 for the Manual revisions.

Subcommittee 9. Vegetation Control

Assignment D9-1-88, is development of a commentary, which is a new assignment. Work is in progress with a 1990 Manual publication date in view.

Subcommittee 10. Geosynthetics

Assignment D10-1-86, dealing with geotextiles in erosion control applications, and Assignment D10-2-86, dealing with drainage applications of geotextiles, were both the subject of a successful letter ballot completed this September.

Assignment D10-3-88, geotextiled stabilized walls and slopes, is a new assignment.

Excerpts From Annual Report of A.R.E.A. Committee 2 - Track Measuring Systems

W.M. Worthington, Chairman

The Committee has six (6) Subcommittees functioning as follows:

Subcommittee 1—Investigate characteristics of corrugation.

Assigned: March 1, 1985

Estimated Completion: June, 1990

The committee presented a paper of its initial findings at the 1987 spring technical conference. Work is now proceeding on a second phase (with a new chairman) using Jackson Jordan data collected on the CP and correlating it with CP Track Geometry Car and their corrugation reference device. Our initial findings were based on small stretches with data collected by hand. This phase will expand the amount of data available for analysis and more accurately relate it to track alignment and track stiffness if possible.

Subcommittee 2—Establish, through an internal survey, the need for track survey data of different types and the requirement of each data element to be updated.

Assigned: March 1, 1985

Estimated Completion: June, 1989

Subcommittee 2 has completed a survey to establish what elements of track data can and should be stored in a computerized database. The subcommittee is now evaluating what data can be collected automatically and what the best methods are for collecting it. The Subcommittee is working on establishing guidelines for collecting data, which will include frequency of data collection, best methods of updating data, restrictions while collecting data and accuracy and resolution of the data.

It was resolved that only data which could be collected automatically would be further investigated. The importance of each of these items will be evaluated along with what each item could be used for, and in what decision making process the data is currently utilized.

The Subcommittee had two presentations at the Automated Track Measurement Symposium in Chicago in March. William Moorehead discussed several existing methods of automated data collection. Antonio Cabrera presented a paper on the development of the New York City Transit Authority's Track Engineering Database.

Subcommittee 3—Evaluate the need and potential use by railways of information on track modulus, subgrade moisture and lateral tie strength, each of which can be evaluated with current technology.

Assigned: March 1, 1985

Estimated Completion: June, 1990

This assignment was initiated in 1985 and has been worked on by the subcommittee under the leadership of three chairmen to date. In order to evaluate the need by Railways of the three aspects of track and structure mentioned in the assignment as well as other possible areas of interest, two questionnaires were prepared and sent out to various railways, suppliers/consultants and governmental agencies. The questions developed for the questionnaires were done so using twelve (12) possible areas of interest that may be needed by a user railway and the availability of devices or technologies which may supply a solution to the need. Each possible device was explained and the respondents were asked to comment on the need, or viability of such a product, device or measurement. The results were then ranked in order of interest shown.

The results of this ranking were then presented to the committee and interested parties as part of a paper prepared for the March 16, 1988 - "Symposium on Automated Track Measurement" in Chicago. This brings the assignment to approximately 90% completion having only to prepare a final document of our findings and present it to the general committee. This is expected to be ready by spring of 1989. The results of the assignment show where, in the field of new technology, the greatest benefits to the Railway Industry are today. The three most needed areas of investigation are:

1. Rail wear measurement
2. Lateral rail stress analysis
3. Rail flaw detection

We not only recommend the continuation of the present assignment to completion but also the mandate to delve into a new assignment to investigate the three areas of Railway needs arising out of these questionnaires.

Subcommittee 4—Propose recommended practice for geometry cars.

Assigned: March 1, 1985

Estimated Completion: June, 1990

To complete its assignment the Subcommittee compiled an inventory of track geometry cars in use in the Free World. This inventory contained specifics on this measuring equipment in use on these cars, the type of car and its capabilities. Advertisement of this inventory was made in RT&S and enjoyed a strong response for copies. Current plans call for updating this survey.

In compiling the survey, a need was seen to redefine certain track terms to bridge terminology differences amongst suppliers and users of track geometry cars and equipment. A letter balloting procedure is now underway to arrive at a consensus on these definitions. We hope to publish the new definitions in the 1990 bulletin.

Two members of the subcommittee, R.N. Harbuck and R.L. Engle of the Santa Fe made presentations at the symposium of Automated Track Measurement in March 1988.

Subcommittee 5—Survey Data Processing Techniques and equipment used on different railway's Track Geometry Vehicles.

Assigned: May 1, 1985

Estimated Completion: June, 1989

Meetings were held in Philadelphia to compile general criteria to determine what constitutes a Track Quality Index.

The purpose of such criteria is to enable each railroad to translate track quality indices computed by other railroads in terms of its own techniques. This would avoid a "lock step" method to assess track quality.

Additional meetings will be scheduled to poll subcommittee members on any additions or deletions of the criteria. Detailed minutes consisting of 2 years effort have been misplaced. Every effort will be undertaken to reconstruct the information.

Subcommittee 6—(a) Develop a recommended practice describing minimum acceptable flaw detection system capabilities.

(b) Develop a recommended practice using rail specimens to stimulate defects of known size and orientation to demonstrate minimum rail flaw detection system capabilities.

(c) Interface with organizations involved in the development of rail inspection technologies.

(d) Inform the membership on the state-of-the art in rail flaw detection.

Assigned: June 1, 1986

Estimated Completion: June, 1990

A draft specification of recommended practice for inclusion in Ch. 2 has been written which specifies the minimum acceptable testing reliability in terms of flaw type and size. It has been analyzed by the Transportation Systems Center and determined to be consistent with the capabilities of the current equipment. Upon review by Mr. T. Ferguson of the UP, the specifications were tightened. These were then presented to all available testing contractors, who confirmed that they could meet the tighter specs. The latest draft is being reviewed by the subcommittee prior to submission for AREA approval.

Development of a recommended practice using test rail specimens has just been started. The subcommittee is reviewing the specifications used by European railways, while canvassing opinion on the feasibility for North American railways.

Attempts have been made to establish a working relationship with CORT, the Committee on rail testing. Mr. McCown, the previous chairman, attended a meeting in 1987 and was met with suspicion over the role of AREA in rail testing, albeit much of this was undoubtedly due to his FRA affiliation. Mr. Roney, the current chairman, and a railway representative, will attend CORT in February 1989 to attempt to re-establish cooperation.

Mr. Roney presented a paper at the March AREA 2 seminar on Automated Inspection entitled "Rail Flaw Detection; Where is it Headed"? This paper was intended to fulfill the current requirement for education of the community on opportunities and pitfalls with rail flaw detection.

Problem Areas: Poor attendance and a fluid membership on this committee means that little can be accomplished at our regular meetings.

Best future avenue may be in the education area where the committee is in a good position to disseminate information generated through CORT and through European and Australian practice to the railway community as a whole.

Excerpts From Annual Report of A.R.E.A. Committee 3 - Ties and Wood Preservation

C.L. Hardy, Chairman

Status of Subcommittee Assignments:

Subcommittee A—Recommendations for further study and research.

No formal recommendations for further study at this time; however, we continue to monitor imported timber for use as cross ties and developments in treatment.

Subcommittee B—Revision of Manual.

While we continue to monitor the need for manual revisions, none are necessary at this time.

Subcommittee No. 1—Cross and Switch Ties.

Assignment dates 1963. The specifications for ties are kept up to date and we continue to investigate design and spacing revisions; however, we have no significant changes to report.

Subcommittee No. 2—Wood Preservatives and Preservative Treatment of Forest Products.

Assignment dates 1963. The specifications for preservatives, as well as specifications for seasoning, conditioning, treatment and handling are kept up to date and research in these areas is monitored. This is a very important assignment since environmental problems continue to be present. We watch closely the development of new types of preservatives and treatments.

Subcommittee No. 3—Service Records of Forest Products.

Assignment dates 1963. We continue to furnish information concerning annual tie renewal and gather information on service test records. This function is most important at a time of railroad consolidation and headquarter relocations.

Subcommittee No. 4—Collaborate with AAR Research Department and Other Organizations in Research and Other Matters of Mutual Interest.

Assignment dates vary from 1963 to 1982. We continue to work with AAR and other organizations in each segment of this assignment. Reconstituted ties are becoming widely accepted in certain areas and anti-splitting devices are again being used in significant numbers. There is considerable research on iron degradation. The relationship between AREA and AAR is stronger now than in the past.

All subcommittee assignments listed above are continuing at this time.

After our extensive modifications to the subcommittee assignments last year, we have no further recommendations for changes.

Excerpts From Annual Report of A.R.E.A. Committee 4 - Rail

A.W. Worth, Chairman

Brief status of each Subcommittee Assignment:

Subcommittee A

Subcommittee A is assigned with the recommendations for further study and research and is a continuing committee.

This committee has assigned all recommended subjects for study to the appropriate subcommittees.

Subcommittee B. Revision of Manual.

A continuing committee. All the requested manual revisions have been assigned to the appropriate subcommittees.

Subcommittee 1

C1-1-74. Collaborate with technical representatives of rail suppliers, welding contractors, suppliers of field welding, rail grinding and rail testing contractors on matters of mutual interest.

A continuing committee, necessary to maintain liaison between railroad members and suppliers and keep all concerned informed of ongoing improvements and changes in technology.

D1-3-83. Ad Hoc Committee on Rail Profile Grinding

Assigned 1983. Final report has been made and approved by vote of the main committee. It will be published in the December bulletin and subcommittee chairman A. M. Zarembski is expected to present it at the Annual Technical Conference in March 1989.

D1-2-81. Qualification Testing

Assigned approximately 1981. Ongoing; due to nature of subject, no definite date of completion is evident at this time. Metallurgical research work in is progress at AAR. Committee has not reported recently and will be asked for an update. Recommend subject be continued. Project involves means of ensuring ongoing quality control of welds at the welding plants, desirability of which is self-evident.

D1-1-81. High Strength Rail Welding

Assigned approximately 1981 (or earlier). When first started this subcommittee concentrated on welding techniques for chrome alloy rails, but the chairman resigned before any report was submitted and the subcommittee became inactive. Committee was re-started 1985 and since then has concentrated mostly on welding heat treated rails. Purpose of subcommittee is to develop and standardize techniques for making welds in high strength rail that will not be subject to brittleness, batter or spalling. Chairman expects to be able to start writing final report around the end of this year for publication in 1989. A very worthwhile subject; recommend be continued to completion.

Subcommittee 2

C2-1-55. Collaborate with technical representatives of rail and joint bar manufacturers in research and other matters of mutual interest.

A continuing committee, necessary to maintain liaison between railroad members and rail and joint bar manufacturers and keep all concerned informed of ongoing improvements and changes in technology. Research work is ongoing on ultrasonic testing, stamping, side sweep, long rail lengths and tolerances.

D2-1-88. Web Cracking of Rail from Roller Straightening

This is the AREA designation of an ad hoc subcommittee set up between the AAR, the North American steel mills and a number of Canadian and U.S. railroads in January 1988 on an emergency basis, to investigate causes of split webs in newly received rails from a number of different mills. If defective rails get into track they not only are expensive to replace but present a derailment hazard. Subcommittee met most recently in Chicago July 21 to discuss phenomena with Sarah T. Wineman of MIT, who has written a paper on the subject. It is too early yet to forecast date of completion, but study should be continued.

D2-2-88. Tolerance and Method of Measuring Rail Head Radius

Assigned May 10, 1988. As this subcommittee is still in process of being organized, there are no interim reports as yet and no completion date. Subcommittee was set up because many railroads commented that though designs for rail head radii on AREA plans are based on rather precise

assumptions as to wheel-rail contact the head radii measured in new as-rolled rails are likely to be quite different. Differences in head radii affect contact stresses in wheels and rails and affect wheel tracking. Recommend study be continued.

Subcommittee 3

C3-1-76. Rail Statistics

Assigned 1976. This committee reports in a standardized format each year the rail shipped to North American railroads from North American and non-North-American producing mills, by weight and section. The first report in the new standard format (for 1985) was published in the October 1987 AREA bulletin. Reports for subsequent years will be published annually thereafter. A continuing committee; recommendation is that subject be continued.

Subcommittee 4

C4-1-73. Update data on methods and equipment for making welding repairs to rail and turnouts including thermite welding.

A continuing committee. This committee maintains liaison with the AWS, which is active in the field of trackwork welding, and with AAR in connection with research at the FAST loop. Discussions are in preliminary stages on a possible review of present AREA recommended practices for thermite welding, as published in the Manual, to reduce hazard from torch cut rail ends. Recommend subject be continued.

Subcommittee 5

C5-1-78. Rail Specifications, Research and Development.

This is a very important continuing committee. It has just completed a major revision to the Manual, Chapter 4, to incorporate ultrasonic testing standards and macroetch standards for rail, to be published this year. A further revision for 1988 will be the drawing for the 133 RE rail section. Subcommittee 5 is vital to the functioning of the Committee and it is recommended the subject be continued.

Subcommittee 5 did not present a report at the May 1988 meeting and its Chairman has since resigned due to change in duties. It is expected a new Chairman will be named at the meeting in Pueblo in October 1988.

D5-1-84. Recomputation of Physical Properties of Rail Sections

Physical properties of rail sections now published in the Manual were computed by hand many years ago and when they are checked by modern mathematical computer techniques they have been found to be inexact. Now that such techniques are available the Committee suggested in 1984 that they be worked out for all modern rail sections. Work is 99% complete on calculations but can not be completed awaiting advice as to constant to be used to convert end area to weight. We recommend study be continued. Results will have to be balloted to the whole committee for approval before publication. Date of completion depends on getting a satisfactory density for rail steel, and on ability to get someone to do drawings of satisfactory quality to AREA standard.

D5-2-88. Wheel Rail Contact Stresses

This is the AREA designation of an ad hoc subcommittee set up early in 1988 between the AAR Engineering and Mechanical divisions and a number of major North American railroads to study (a) what recommendation should be made for a new standard AAR wheel profile and (b) what changes to standard AREA new rail head profiles and rail profile grinding techniques might be necessary to produce optimum rail life with such new wheel profiles. This is a co-operative committee being pursued primarily through the AAR and we recommend it be continued. It is too early as yet to forecast a date of completion.

Subcommittee 6

C6-1-62. Joint bars, design specifications, service tests, including insulated joints and compromise joints.

A continuing committee. Currently this committee is developing specifications for the fabrication of bonded insulated rail joints. It also has undertaken a complete review of all the current drawings for splice bars now in the manual, as consensus is that the current drawings are out of date. Now that the 133 RE section has been adopted as standard, the subcommittee has been asked to give priority to adoption of a drawing or drawings for 133 RE splice bars. The subcommittee now has a new chairman, who will make a progress report at the October 1988 meeting in Pueblo.

Subcommittee 7

C7-1-68. Effect of Heavy Wheel Loads on Rail

A continuing committee. This committee has an ongoing study related to the effects of heavier cars on the fatigue life of rail steel. It handles the liaison, for example, with AAR on rail fatigue life prediction computer models and on results from the present HAL test at the AAR Transportation Test Facility. This is an important committee and should be continued.

Subcommittee 8

C8-1-88. Management of Rail Test Resources

Until this year the title of Subcommittee 8 was "Recommendations for interval of non-destructive testing for internal defects of rail and track". The subcommittee was originally set up in the 1970's with the idea that it would be a temporary assignment, to be completed around 1980. As time progressed it became evident that there were too many variables and intangibles to permit anything definite to be published in simple tabular form, and approval has now been given for a change in title and assignment to what is shown above.

The newly retitled Subcommittee did not present a report at the May 1988 meeting. It will be asked for a work plan at the October meeting in Pueblo. It is recommended that this subject be continued, and that the Subcommittee be designated as a continuing subcommittee.

Excerpts From Annual Report of A.R.E.A. Committee 5 - Track

W.B. Dwinnell III, Chairman

Subcommittee Assignments:

A. Recommendations for Further Study and Research

Committee has communicated with AREA staff concerning method to change drawings in the portfolio.

B. Revision of Manual

B-1. Review anchor pattern for bolted rail and CWR subject completed.

B-2. Review current anchor application and maintenance procedures in collaboration with Subcommittee 7. Responses from questionnaire were received and work continuing on this topic.

B-3. Response to questionnaire on "Recommended Practices for Preservation of Track Fixtures" have been received. Based on responses, the Committee has decided not to discontinue working on the subject.

2. Track Tools

C2-1-79. Review of new and additional track tools.

Letter ballot on "Insulated Track Tools" has been approved and forwarded to AREA staff for inclusion in the Manual.

Currently working on specifications for one-man aluminum tie tongs and magnaflux testing of impact tools.

4. Track Design Collaborating as Necessary or Desirable with Committees 1, 3 and 4.

D4-1-84. Elastic rail fasteners - wood ties.

Letter ballot was taken and did not pass. Subcommittee is reviewing and revising for later submittal. Still a very active subject.

D4-2-84. Hold-down fastener - wood ties.

This is being reviewed in conjunction with D4-1-84.

D4-3-85. Tie plates - wood ties.

Subject is active and subcommittee is reviewing a draft submittal.

5. Turnout and Crossing Design, Collaborating as Necessary or Desirable with Committees 1, 3 and 4.

Eight recommended changes were submitted for letter ballot and six passed for changes in the Portfolio of Trackwork Plans.

D5-1-77. Review of guard rails protecting frogs. Letter ballots submitted and approved.

D5-3-80. Investigate use of fastening agents for track work. Subject is being reviewed.

D5-2-79. Investigate use of gage plates on turnouts to maintain gage. Subject is currently being reviewed.

D5-4-81. Develop specification for explosion hardening of track work castings. Subject is still under discussion and initial letter ballot approved.

D5-5-81. Develop recommended maintenance parameters for frogs, switches, guard rails and crossings. Drawings are under current review.

D5-6-88. Study use of direct fixation fasteners on turnouts. Work has begun and initial report being discussed.

This subcommittee has voted to invite various suppliers to make presentations on turnout technology being used in other countries during the Spring 1989 meeting. This proposal has been forwarded to AREA headquarters for input.

7. Track Maintenance Collaborates as Necessary or Desirable with Committees 1, 3, 4 and 22.

D7-1-84. Study rail lubrication. Questionnaire has been prepared and being distributed.

D7-2-87. Maintenance parameters for frogs and switches. This subject is being handled by Subcommittee 5 and should be shown there.

D7-3-87. Anchor application and maintenance procedures. This subject is complete.

8. Criteria for Track Geometry Design

D8-1-84. Ballast Shoulder Width

Committee 1 is considering a Manual revision to recommend the use of 12-inch shoulder width for CWR and a 6-inch shoulder width for jointed rail. Voted to endorse these changes by Committee 1.

D8-2-84. Underbalance of Superelevation and Effects on Track.

Discussion centered on the underbalance and maximum elevation. Most freight railroads use an average of 3" underbalance with a maximum of 6" elevation. One railroad is studying the effects of using 4" underbalance with a 9" maximum elevation; will be reviewed further for discussion at the next meeting.

D8-3-85. Effects of Lack of Superelevation in Turnouts and Crossovers

Committee was advised that AAR may be doing further tests on speeds through level turnouts. This subject will focus on allowable speeds through level turnouts. This may involve an expansion of the Manual material to explain the basis for the maximum allowable speeds recommended. Study of this subject will continue.

Excerpts From Annual Report of A.R.E.A. Committee 6 - Buildings

D.V. Fraser, Chairman

Subcommittee Assignments:

D-2-81. Design Criteria for CTC Dispatching Facilities

A draft of this report is under refinement and it is planned that the report will be ready for final reading at our January 1989 meeting.

D-5-88. Design Criteria for wheel and bearing shops.

A draft of this report is under further review and the report should be ready for final reading at our January 1989 meeting.

D-3-84. Railway Office Buildings and **D-4-85 Freight Car Repair Shop** are being reassigned due to the work demands on their former chairmen. Subcommittee **C-1-77 Architectural Competitions** has been abolished. We also voted to establish a new Subcommittee to be named Design Criteria for Material Storehouse and Distribution. A chairman and designation for this Subcommittee will be established by the secretary after discussions with AREA staff.

It was also moved seconded, and carried that Subcommittee report Part 6 Locomotive Sanding be changed to Design Criteria for Locomotive Servicing Facilities. The work of Subcommittee **D-1-80 Building Design for Energy Conservation** is at the AREA awaiting publication as a 1989 Manual revision.

Excerpts From Annual Report of Committee 7 - Timber Structures

D.C. Meisner, Chairman

Subcommittee Status:

Assignment A—Recommendations for further study and research

No new projects until some existing ones are closed out. Chairman is researching subjects that would be beneficial.

Assignment B—Revision of Manual

Revised manual printed, possible minor changes in table for working unit stresses for Structural Lumber. We thank the AREA for a good printing of our Chapter.

Assignment C-1-53—Specifications for design of wood bridges and trestles

a. Paper on Current Design Practices for Railroad Timber Trestles. This has been submitted to the AREA for approval to be published as information only.

b. Changes in Design, six items to be considered; it is expected to take about 1 1/2 years to complete. Problem area would be tables to be added for E-80 design. Chairman doing a good job.

Assignment D-4-82—Effect of unit trains on timber trestle components

Two chairmen have been in charge of this committee since 1982. The last chairman has resigned his position with the railroad he worked for. If new chairman is not selected, I recommend this be dropped for lack of support.

Assignment D-3-82—Effect of dapping and end overhang on allowable stresses in bridge ties

Questionnaires have been sent out. When replies received, the results will be published. About six months needed to complete. Committee will then be dropped.

Assignment D-2-74—Protection of pile cut-offs; protection of piling against marine organisms by means other than preservatives

Final report made on part concerning marine organisms and report will be sent out shortly. Report on protection of pile cut-offs is about 40% complete and should be completed in about 9 months.

Assignment D-1-64—Study of in-place preservative treatment of timber trestles

A final report is complete and was just sent out to members. Will be submitted to be published as information only, and a request to drop this committee will be made.

Excerpts From Annual Report of A.R.E.A. Committee 8 - Concrete Structures and Foundations

D.J. Lewis, Chairman

Subcommittee Activities:

Subcommittee A. Recommendations for Further Study and Research

In cooperation with the other two structural committees, the possibilities of further research related to railway bridges is being pursued. A Bridge Research Workshop was held at the University of Illinois, Urbana, IL on October 28 and 29, 1987. A report entitled "National Workshop on Railway Bridge Research Needs: Summary Report" is being drafted and will be published by the AAR.

Subcommittee B. Revision of Manual

This subcommittee is actively reviewing Chapter 8 to make the presentation consistent with the "official" AREA format. All ballot material will be reviewed by this subcommittee prior to the full committee vote.

Subcommittee 1. Design of Concrete Structures

D1-1-84. Develop specifications for precast and cast-in-place concrete segmental bridges.

Work on the segmental bridge specifications was suspended until the Post-Tensioning Institute's recommendations are available.

D1-2-86. Develop criteria for rating existing bridges using load-factor design method.

Although voted upon and approved by the full committee, Part 19 was sent back to subcommittee and revised to reflect rating of bridges by either load-factor design or service load design. The revised Part 19 is presently out to the full committee for ballot.

D1-3-86. Develop specifications for design and construction of precast concrete box culverts.

This part (Part 13) is ready for publication in the bulletin.

D1-4-86. Develop criteria for design of railroad bridges in seismic zones.

Design criteria for railroad bridges in seismic zones is progressing slowly. One reason for the slow progress is, the ad hoc committee is awaiting the possible translation of the "Japanese Railways publication on Seismic Design". To date the table of contents has been translated and is of interest.

D1-5-87. Investigate applicable impact factors.

Applicable impact factors are a continuing concern. Mr. S. Skaberna's paper entitled "A Review of Studies of Impact on Concrete Railway Bridges", has not been published yet and attempts will be made to have it published as information. In addition, a questionnaire was sent to Chief Engineers or Vice Presidents of Engineering of the Class 1 Railroads asking for their current design practice. No results yet.

D1-6-88. Develop revised criteria for minimum reinforcement in bridge columns.

This revision is ready for publication in the bulletin.

D1-7-88. Rewrite Part 2, Reinforced Concrete Design:

This is a new assignment and is expected to take several years since this is one of the major design portions of the chapter. Some updated material (a small amount of the Part) has been reviewed and readied for ballot.

D1-8-88. Rewrite Part 17, Prestressed Concrete Design.

This is a new assignment and is expected to take several years since this is one of the major design portions of the chapter. Some updated material (a small amount of the Part) has been reviewed, balloted and ready for publication.

D1-9-88. Develop specifications for design of slabs-on-grade subject to heavy loads.

This is a new assignment and may be incorporated as part of the Part 2 review (Assignment D1-7-88).

Subcommittee 2. Foundations and Earth Pressures

D2-1-87. Rewrite Part 3, Footing Foundations.

This part is ready for publication in the bulletin.

D2-2-87. Rewrite Part 4, Pile Foundations.

This part is in the process of being rewritten. The rewritten part was not ready for letter ballot by subcommittee at our May meeting but should be ready by our September meeting.

D2-3-87. Rewrite Part 10, Reinforced Concrete Pipe.

This part has passed letter ballot by the committee and should be ready for publication in the bulletin shortly.

D2-4-88. Revise Part 22, Specifications for Subsurface Investigation.

Minor progress was made on this assignment because of major effort placed on other four assignments.

D2-5-88. Rewrite Part 5, Retaining Walls and Abutments.

This part has passed letter ballot by the committee and should be ready for publication in the bulletin shortly.

Subcommittee 3. Durability of Concrete

C3-1-88. Monitor current waterproofing practices.

This is a continuous assignment and just had a few changes incorporated in the 1988 Manual.

D3-1-88. Investigate and report on current methods of protecting concrete.

At present, several different reports are being reviewed. After final review is completed, recommendations will be balloted by the subcommittee. This is a new assignment related to the expansion of this subcommittee's responsibility.

Subcommittee 4. Repair, Restoration and Strengthening of Concrete Structures

D4-1-86. Develop criteria and protection to structures adjacent to railway tracks.

This assignment has passed subcommittee ballot and is ready for full committee ballotting.

D4-2-87. Study AAR bridge program and make recommendations for implementation at FAST.

This assignment is dependent on the results of the Bridge Research Workshop report which is being drafted and will be published by AAR.

D4-3-88. Develop specifications for inspection, evaluation and repair of damaged prestressed concrete.

At present the subcommittee is collecting available information and reviewing this literature for its applicability.

D4-4-88. Develop criteria for prevention of scour damage to piers.

The subcommittee has begun reviewing material in the Manual (located in four different Parts) and will attempt to tie all this information together as a single part.

Excerpts From Annual Report of A.R.E.A. Committee 9 - Highway-Rail Crossings

H.A. Richards, Chairman

Status of Subcommittee Assignments:

Committee 9 and ASTM's Committee D-4 on Road and Pavement Materials have agreed to work together to develop standards and/or recommended practices, for the installation and maintenance of highway-rail crossing surfaces. Committee 9 subcommittees to be actively involved in this project are: **D-1-87**; Foundations for Highway-Rail Grade Crossings, **C-1-87** Grade Crossing Surfaces and **D-2-87** Approaches to Grade Crossings. Kurt Anderson, Union Pacific, Vice Chairman of Committee 9 will coordinate committee activity on this subject.

A statement of work has been prepared, and is now being reviewed, to submit to the Federal Highway Administration for a "pooled fund" study to support this activity. Several states have agreed to support the research if FHWA requests applications from states to participate.

Excerpts From Annual Report of A.R.E.A. Committee 10 - Concrete Ties

S.P. Heath, Chairman

Status of Assignments:

Subcommittee A—Recommendations for Future Study and Research

Do not have any recommendations for future research at this time.

Subcommittee B—Revision of Manual

No recommendations for manual revisions that are not already under study in the other subcommittees.

Subcommittee 1—Flexural Strength

Assignment D1-1-88—Investigate the affect of axle loads on tie requirements.

In the review of Tables I and II which was completed last year and the refinement of a factored design approach which became part of the manual this year, we find this assignment is no longer needed as the affect of axle loads on tie requirements was addressed in developing the factored design approach. Recommend this assignment be dropped as it is no longer needed.

Assignment C1-1-87—Monitor developments in prestressed and reinforced concrete technology which may affect concrete tie requirements.

This is an on-going assignment and this subcommittee has representation on ACI committees to stay informed of any new developments in prestressed or reinforced concrete technology. Recommend this assignment continue.

Subcommittee 2—Switch, Bridge and Crossing Ties

Assignment D2-1-85—Investigate requirements for concrete switch ties, bridge ties and grade crossing ties.

Have not been able to make much progress this year but with more concrete tie turnouts in service, hope to make more progress next year. Recommendations for the manual should be completed in two more years. Recommended procedures and data for designing switch ties, bridge ties and crossing ties are needed in the manual and recommend this assignment be continued.

Subcommittee 3—Fastenings

Assignment D3-1-83—Revise current test requirements.

Portions of these revisions have been discussed in the committee meetings and a draft of proposed manual revisions will be presented at the September meeting. This assignment should be completed within a year.

Assignment D3-2-83—Investigate the affect of axle loads and tie spacing on fastening requirements.

Will be able to put more effort on this assignment when the first assignment is completed and also may be able to get some useful information from the "Heavy Axle Load Program" currently taking place at Pueblo. This investigation should take another three years to complete and determine if manual revisions are needed. Recommend this assignment be continued.

Subcommittee 4—Test Requirements

Assignment D4-1-86—Review and recommend revisions of the load magnitude specified for the fastening repeated - load test.

After preliminary study by Subcommittee 4, this assignment is being referred to Subcommittee 3 as it is directly related to fastener characteristics. Recommend this assignment be continued under Subcommittee 3.

Assignment D4-2-86—Review and recommend revisions of the rail seat overload and ultimate load test for two block ties.

After preliminary study, Subcommittee 4 recommended this be referred to Subcommittee 1 as it primarily concerned the load specified for the rail seat overload and ultimate load test. Subcommittee 1 has reviewed the manual as revised in 1988 and feel the loads specified in the manual for these tests, 1.75 P for two block ties, is the proper load for this test. Recommend this assignment be dropped.

Assignment D4-3-87—Evaluate acceptance criteria for repeated load test.

Work is continuing on this assignment and currently addressing the need to require no-cracking of the tie during this test. A test involving several levels of loading is also being evaluated. This assignment should take two more years to complete and recommend this assignment be continued.

Subcommittee 5—Maintenance Requirements

Assignment C5-1-87—Maintenance requirements of concrete ties, including pads and insulation.

Currently preparing a questionnaire to determine railroads maintenance practices concerning concrete tie track compared to wood tie track. Will also be collaborating with Committee 12 concerning maintenance practices on transit lines. This is an important issue and this assignment should continue.

Subcommittee 6—Ballast Requirements

Assignment C6-1-87—Collaborate with Committee 1 on concrete tie ballast requirements.

This is an on-going assignment and provides a means to get data related to ballast for concrete ties to Committee 1 and should continue.

Excerpts From Annual Report of A.R.E.A. Committee 11 - Engineering Records & Property Accounting

R.M. Davis, Chairman

Subcommittee Assignments:

Subcommittee "A": Recommendations for Further Research and Study

Chairman Ranuio spent much of his time assisting other subcommittee chairman during the year. At the same time, consideration was given to further study on the issue of Depreciation studies that are required by the ICC and reporting detailed property items as stipulated under existing regulations.

Subcommittee "B": Revision of Manual

Technical chapter 11 of the committee was completely rewritten during 1987. Printers proofs were forwarded to the Committee, some necessary corrections were noted and returned to AREA headquarters. The rewrite of Chapter 11 has now been officially sanctioned by Committee 11 and included in the AREA manual.

Subcommittee "1": Accounting

A review of account differences between Canadian and American regulations is being progressed. We are researching this subject with our Canadian members and a report will be prepared spelling out the differences in accounting regulations.

An addition to the manual will result when this subject is finalized.

Subcommittee "2": Office and Drafting Practices

Part 2 of Chapter 11 now contains current state-of-the-art information on automated design and drafting systems. Under the guidance of Vice Chairman Ranuio, Subcommittee 2 has instituted research of current systems on all class 1 railroads.

Subcommittee "3": Taxes

The analysis of the major ramifications of federal tax legislation is continuing. It is still Committee 11's intent to make a presentation on the impact of this legislation at a future AREA technical conference. FAS 96, revised accounting treatment of deferred taxes, presents new complications in this area.

Subcommittee "4": Planning, Budgeting and Controls

The adoption of this function by Committee 11 has generated a great deal of enthusiasm in our committee. It has also generated a diversity of membership from railroads and outside consulting firms. Subcommittee 4 continues with their research on methods and procedures. We are reviewing comments sent to us from the Chairman of Committee 22 who has expressed concern that an overlap of responsibilities may exist between Committee 11 and Committee 22. We have solicited comments from our members and will reply to AREA headquarters shortly.

Excerpts From Annual Report of A.R.E.A. Committee 12 - Rail Transit

D.W. Reagan, Chairman

Subcommittee Assignments:

Each subcommittee is preparing a draft chapter outline for review at the fall meeting in October at San Francisco. Prior to the October meeting, subcommittee activity was as follows:

Subcommittee 1—Rail Corridor Evaluation

The new chairman reported that the outline for corridor design and planning was reviewed with some additions made. The subcommittee will improve communications by using phone conferencing and telefaxing.

Subcommittee 2—Special Trackwork and Roadway Considerations

The subcommittee had continued progress with their questionaire and developed a data analysis group. The committee is considering splitting subcommittee 2.

Subcommittee 3—Special Bridge and Structural Considerations

The subcommittee developed a prioritized list of facilities to be considered in rail transit structures.

Subcommittee 4—Rail Transit Electrification, Collaborating as Necessary with Committee 33

This subcommittee assignment was approved by the AREA Board at their June, 1988 meeting. Chairman Reagan noted the following:

Committee 12 to solicit membership for qualified electrical people to work on Subcommittee 4. Recruiting will deal with both catnery and contact rail, and collaborating as necessary with Committee 33.

Excerpts From Annual Report of A.R.E.A. Committee 13 - Environmental Engineering

R.C. Brownlee, Chairman

Status of Subcommittee Assignments:

Subcommittee 1—Water Pollution Control

Assignments continue to monitor evolving regulations on underground storage tanks and stormwater runoff, Assignments D-1-1-85 and D-1-2-86 respectively. Assignment "Remediation of Contaminated Groundwater" is being conducted jointly with assignment D-1-3-88 of Subcommittee 3 which addresses solid waste management. Work has been slow due to lack of response from Subcommittee members.

Subcommittee 2—Air Pollution Control

Work continues on the 1985 assignment on services of Part 2 of Manual material. The Subcommittee Chairman has assumed duties of Committee Secretary. A new Subcommittee Chairman is being sought for completion of the assignment.

Subcommittee 3—Land Pollution and Solid Waste Management

The 1985 assignment for revision of Part 3 of Manual material has progressed with approval for publication as Information in the Bulletin a portion of the revision titled "Solid and Hazardous Waste Management - An Overview of Regulations." Subcommittee 3 is now working with Subcommittee 1 on control of groundwater contamination.

Subcommittee 4—Noise Pollution Control

Assignment D-4-86 "Report on Noise Barrier Technology" was approved for inclusion in the 1988-89 Manual Supplement. The supplement was published in July 1988. The Subcommittee is now inactive and members have been reassigned.

Subcommittee 5—Plant Utilities

Assignment D5-1-86, Revision of Part 5 Material in Manual with Incorporation of Section of Former Part 4 and Part 5, is continuing but slowed by lack of response by members. Much of the material is no longer relevant to the charge of the Committee and will be deleted. Assignment D5-2-86, Environmental Considerations for Locomotive Fueling Facilities and Devices is slow due to lack of response.

Excerpts From Annual Report of A.R.E.A. Committee 14 - Yards and Terminals

W.A. Schoelwer, Chairman

Status of Subcommittee Assignments:

Subcommittee A

One new item - Rail Water Facilities has been requested as a subcommittee item for development.

Subcommittee B

An update of Part 4.3 of Chapter 14 (Automobile and Truck Transport) has been approved as manual material. Voting is presently in progress.

Subcommittee C-1-82—Intermodal Terminals

The original report has been published in the manual. The subcommittee will be updating the manual material on a periodic basis. Presently this subcommittee is working on incorporating the handling of doublestack equipment.

Subcommittee C-2-88—Working with TRB on Intermodal Terminals

This is an on-going effort to coordinate their recommendations to be sure they meet AREA guidelines. This is presently a one-man committee with little going on but we still feel some contact is required.

Subcommittee D-1-74—Bulk Material Handling System

The report has been published as information and is now being modified to be submitted as manual material. We expect to have the material ready early next year.

Subcommittee D-3-84—Run Through Trains, Effect on Yards

This subcommittee began work in 1983 and went through four chairmen. The present chairman has progressed the work and we expect to submit a report for information next year.

Subcommittee D-4-86—Control of Cars with Contaminated Wheels

The subcommittee has made some progress and a preliminary report will be submitted in October 1988. A report for information will be published in 1989.

Subcommittee D-5-86—Design of Automobile Loading/Unloading Terminals

This subcommittee has completed its work and the material is being voted on at present. We expect to submit it to the Board of Directors in October.

Subcommittee D-6-87—Design of Reclamation Plants

A preliminary report has been received by the full committee and a revised report will be discussed at the next meeting. The final report will be submitted as information in early 1989.

Subcommittee D-7-87—Yard Control Systems

The subcommittee chairman has not submitted any report since committee inception. However, a first draft is partially complete and will be reviewed at our October 1988 meeting.

Subcommittee D-8-88—Bulk Fluid Transfer Facilities

This subcommittee has just begun to work. A preliminary report is expected in early 1989. A final report as information is expected in 1990 and updating the manual in 1991.

Excerpts From Annual Report of A.R.E.A. Committee 15 - Steel Structures

E. Bond, Chairman

Brief status of subcommittee assignments:

A. Recommendations for further study and research

The Committee hopes to work closely with the AAR bridge research project by working through John Choros of the AAR lab.

B. Revision of Manual

The committee has been working to prepare this year's submittal for changes to chapter 15 of the AREA Manual and is working closely with other subcommittees on proposed future changes.

C-1-60. Bibliography and technical explanation of various requirements in AREA specifications relating to iron and steel structures

There is a continual need for this committee to update and add items in its area of responsibility.

C-2-82. Fracture control plan

This subcommittee has developed a fracture control plan and has obtained approval for Manual changes in this area which will be submitted in October. It needs to continue on a temporary basis as additional minor modifications are made.

C-3-86. Steel fabrication - materials, methods, quality control procedures and qualifications of fabricators

The committee is working on the development of specifications for loading details for fabricated members.

D-1-79. Develop specifications for the earthquake design of steel railway bridges

The subcommittee will submit it's recommendation for an earthquake design section to be added to the manual, this October.

D-2-87. Develop specifications for the design of confined elastomeric bearings in collaboration with Committee 8

The committee continues work on developing specifications for Manual inclusion.

D-3-87. Obtain data from which the frequency of occurrence of maximum stress in steel railway bridges may be determined under service loading

This committee has a very difficult task and has been working with the bridge rating group. More test data is needed and it is hoped that some of this will come from the AAR Bridge Research project.

D-4-88. Methods for repairing damaged steel bridge members

This is a new committee and work has begun by collecting examples of damage and methods of repair.

Excerpts From Annual Report of A.R.E.A. Committee 16 - Economics of Plant, Equipment and Operations

C. Bach, Chairman

Brief Status of each subcommittee assignment:

Subcommittee B: Revision of Manual

Revisions are planned to be made to the Train Resistance formula (Davis equation) in order to incorporate recently developed computer models such as the AAR Train Energy Model. Rail life prediction section is also under review. This is a continuing assignment.

Subcommittee 1:

D-1-84. Economics of Train Speed

The report has been approved by the Committee and will be submitted for publication.

Subcommittee 2:

D2-1-83. Economics of Automatic Train Inspection Equipment and Location Including Consideration of Unattended Rear of Trains

AAR study will be reviewed to see whether it can be used in this assignment. To date, surveys have resulted in data that was not suitable for economic analysis. Use of automatic train inspection equipment could result in substantial reduction of operating costs. Continuation and completion is dependent on whether suitable economic data can be obtained for this assignment.

D2-2-83. Economic Comparison of Track and Right-of-Way Inspection Methods and Equipment

A recommendation was made at the last Committee meeting to drop this assignment, due to the complexity and sensitivity of material required to carry out this assignment.

Subcommittee 3: Application of Industrial Engineering to the Railway Industry

D3-1-85. Application of Robotics in the Railway Industry

The final report was published in Bulletin 716, May 1988. No further work will be done on this assignment, and it is recommended that this assignment be replaced with a new Industrial Engineering assignment.

D3-2-87. Railway Application of Artificial Intelligence

Contrasting with limited possible application of robotics in railways, as found in the previous assignment, artificial intelligence is out of the research area and well into the application area. This was demonstrated at the last Committee meeting when presentations from railways were given. Significant benefits had been obtained by these railways in areas where AI had been applied. The Committee is working closely with AAR on this assignment through the recently appointed AAR liaison representative. Research required is quite extensive for this assignment and a final report is estimated to be completed by December 1990.

Subcommittee 4:

D4-1-87. Economics of Railway Operations without Institutional Restraint

Considerable information has been gathered, and current changing conditions pertinent to this assignment are being closely monitored. Institutional restraints prevent railways to take advantage of available technology to gain reduction in operating costs. This assignment plans to study the economics that might be gained if railways were free to operate without many of the present restraints. The final report is expected to be completed by December 1990.

Subcommittee 5:

D5-1-87. Factors to be Considered in Evaluating Advanced Train Control System

A list of factors has been prepared and submitted to the Committee for review. This assignment plans to identify factors that should be considered when evaluating the benefits to railways of ATCS. Good progress has been made and the final report is expected to be completed by December 1989.

A.R.E.A. Committee 17 - High Speed Rail

The first meeting of this committee was held on November 3, 1988 in Washington, D.C. The following responsibilities and subcommittee structure were adopted and discussed at the meeting:

This committee is responsible for the development and publication of information and recommended practices regarding wheel-rail systems capable of operating in the 150-250 mph speed range, and also reporting on developments which would permit operation of such systems above 250 mph.

Subcommittee structure:

A. Recommendations for further study and research.

B. Proposed Manual Chapter.

D-1-88. High Speed Rail Corridor evaluation, including Terminal considerations.

C-1-88. Track considerations for High Speed Rail, including loads on structures, collaborating with appropriate vehicle groups and committees 4 & 5.

C-2-88. Signal and Electrification considerations for High Speed Rail, collaborating with appropriate vehicle groups and committee 33

D-2-88. Evaluation of systems for operation over 250 mph.

Excerpts From Annual Report of A.R.E.A. Committee 22 - Economics of Railway Construction & Maintenance

W.C. Thompson, Chairman

Status of Subcommittee assignments:

Subcommittee B, Revision of Manual

(a) Assigned - Unknown.

(b) Progress - Chairman 22 Rewrite of Manual underway with progress of each part shown below.

(c) Completion date: 6/92 per Committee's five-year goals.

(d) Benefits - Greatly improved quality of information in Chapter 22 of Manual.

(e) Problem Areas - None.

(f) Recommendations continue.

Subcommittee B-1, Part 1

Revisions of Manual - Personnel

(a) Assigned - 1/14/88.

(b) Progress 5% - Initial meeting held.

(c) Completion Date - 6/91.

(d) Benefits - Improved information.

(e) Problems - None.

(f) Recommendations continue.

Subcommittee B-1, Part 2

Revisions of Manual - Programming Work and Budgeting

(a) Assigned - 1/14/88.

(b) Progress 30% - Subcommittee met June, 1988, in Lake Louise to develop plan of attack. Will delete portion of Part 2 that should be handled by Committee 11.

(c) Completion Date - 1/91.

(d) Benefits - Provide high quality information.

(e) Problems - None.

(f) Recommendations continue.

Subcommittee B-1, Part 3

Revisions of Manual - Construction and Maintenance Operations

(a) Assigned - 1/14/88.

(b) Progress 0% - None.

(c) Completion Date - 6/91.

(d) Benefits - Improved information.

(e) Problems - Need to hold first meeting.

(f) Recommendations continue.

Subcommittee B-1, Part 4

Revisions of Manual - Equated Mileage Parameters

(a) Assigned - 1/14/88.

(b) Progress 20% - Subcommittee met 6/88, in Lake Louise to develop plan.

(c) Completion Date - 6/91.

(d) Benefits - Improved usefulness and understanding of equated mileage parameters.

(e) Problems - None.

(f) Recommendations continue.

Subcommittee D-1-87. Develop Economics of Methods to dispose of scrap and obsolete materials.

(a) Assigned - 1/26/88.

(b) Progress - 80%.

(c) Completion Date - Was 6/88; now 6/89.

(d) Benefits - Addresses an important and extensive topic.

(e) Problems - Delayed. Draft report complete.

(f) Recommendations continue.

Subcommittee D-2-86. Economics of Various Surface Gang Consists used by Railroads in North America.

(a) Assigned - 1/14/86.

(b) Progress - 10%.

(c) Completion Date - Was 1/89; New date to be established.

(d) Benefits - Information.

(e) Problems - Chairman inactive, Resigned.

(f) Recommendations continue to completion.

Subcommittee D-3-88. Economics of Vegetation Control Methods Working with Committees 1 and 13.

(a) Assigned - 1/14/88.

(b) Progress - 10%, new assignment.

(c) Completion Date - Was 6/87; Now 6/90.

(d) Benefits - Develop costs of Vegetation Control Methods.

(e) Problems - None.

(f) Recommendations continue.

Subcommittee D-4-87. Economics of AREA Standard vs. High Strength Rail.

(a) Assigned - 6/87.

(b) Progress - 40%, Questionnaire complete. Approved by AAR Legal and being sent to Committee 22 for response. American Railway Engineering Association Executive Committee to review prior to distribution to Chief Engineers.

(c) Completion Date - Was 1/89; now 6/89.

(d) Benefits - Develop rail usage priority and economics.

(e) Problems - None.

(f) Recommendations continue.

Subcommittee D-5-87. Economics of Standardization of Turnout Materials.

(a) Assigned - 1/87.

(b) Progress - 30% complete. Questionnaire forwarded to Executive Committee for review and approved with comments.

(c) Completion Date - Was 6/88; Now 6/89.

(d) Benefits - Can have significant impact on industry.

(e) Problems - None.

(f) Recommendations continue.

Subcommittee D-6-88. Economics of Rail Grinding

(a) Assigned - 1/14/88.

(b) Progress - Will present with panel at Spring AREA Meeting.

(c) Completion Date - 3/89.

(d) Benefits - Present latest information on Rail Grinding.

(e) Problems - None.

(f) Recommendations continue.

Subcommittee D-7-88. Economics of Various Fixations of Rail to Wood Ties

(a) Assigned - 1/14/86.

(b) Progress - 99%, Study submitted for publication.

(c) Completion Date - 6/88.

(d) Benefits - Review and analyze impact of various fastening systems.

(e) Problems - None.

(f) Recommendations complete.

Excerpts From Annual Report of A.R.E.A. Committee 24 - Engineering Education

C.E. Ekberg, Chairman

Subcommittee Assignments

Subcommittee A. Recommendations for further study and research

Assigned: Prior to 1983

Progress: A proposal was drafted for an AREA Undergraduate Research Fellowship Program. This proposal was subsequently approved by the AREA Board of Directors, and up to two awards in the amount of $4000 each will be made during the 1988-89 academic year.

Completion: Should be continuous.

Benefits: Develop meaningful new assignments.

Problems: None.

Recommendations: Continue.

Subcommittee C-1-82 - Recruiting and Speakers

Assigned: Prior to 1983.

Progress: Publish an annual survey of college graduate hiring by railroad engineering and maintenance departments. Advise schools of the availability of railroad speakers for student groups.

Completion: Should be continuous.

Benefits: Provide starting salary information to the railroads. Inform top students of career opportunities in railroading.

Problems: None.

Recommendations: Continue.

Subcommittee C-2-84 - Faculty Support

Assigned: 1983

Progress: The purpose of this subcommittee is to assist engineering schools who wish to obtain technical material from railroads or consultants for classroom use. The results of this effort have been very limited.

Completion: Committee 24 has decided to discontinue the activities of Subcommittee C-2-84, and to re-direct its' objectives.

Subcommittee C-3-85 - Curriculum Development

Assigned: 1985

Progress: Progress has been delayed due to the successive resignations of two previous chairmen. The purpose of the subcommittee is to provide opportunity for the railway industry to assist in the development of engineering curricula which might relate to railroad needs.

Completion: Probably 1990.

Benefits: Opportunity to develop increased interest in railway engineering on the part of engineering faculty.

Problems: Slow start.

Recommendations: Continue.

Subcommittee C-4-86 - Student Relations

Assigned: Prior to 1983.

Progress: Successfully handled the 1988 student scholarship program through which three engineering students received awards in the amount of $2000 each. A total of 42 applications were received and evaluated by nine judges.

Completion: The scholarship program will be replaced by the Undergraduate Research Fellowship Program. Other objectives of the subcommittee will be pursued more actively.

Benefits: Increased student and faculty awareness of AREA and the railroad industry.

Problems: Stimulation of interest in AREA Student Membership has been slow.

Recommendations: Continue.

Subcommittee C-5-71 Continuing Education

Assigned: Prior to 1983

Progress: Exploring new alternatives to stimulate AREA participation in continuing education.

Completion: Should be continuous.

Benefits: Provides ample opportunity for AREA members to update their knowledge of subject matter which is relevant to the railroad industry.

Problems: Must develop a mechanism for reaching objectives.

Recommendations: Continue

Excerpts From Annual Report of A.R.E.A. Committee 27 - Maintenance Of Way

S.F. Mills, Chairman

Subcommittee assignments:

Subcommittee A — Recommendation for further study and research.

After approval by the board of directors this subcommittee has been combined with subcommittee 8, future needs of machinery. The task of this subcommittee will be to direct the committee towards new tasks, set goals, and interface with the REMSA group.

Subcommittee B — Revision of Manual

Specifications for on-track roadway machines have been completed and sent to the board for approval, it is expected that they will be published in 1989. The next phase of the manual clean-up is underway, with each item in the table of contents being updated.

Subcommittee C-1-74 — Reliability Engineering, as applicable to work equipment.

A questionnaire approved by the board has been sent to all major railroads and replies tabulated, as well the questionnaire has also been sent to REMSA members and those replies are now being tabulated. The report is expected to be sent to headquarters for approval year end. The areas of concern highlighted in the report will determine the future direction of this subcommittee.

Subcommittee C-2-83 — Preventative maintenance of maintenance of way equipment.

Data is being gathered by this subcommittee, from all committee members with the intent of publishing preventative maintenance schedules for major pieces of work equipment.

Subcommittee D-2-86 — Computer applications as applicable to work equipment.

Data on project scheduling and costing was handed out to committee members and discussed, as well the committee was given a presentation of Canadian Nationals work equipment information system and viewed the system in real time operation in the Winnipeg facility. This subcommittee will continue to gather information on computer assist packages for presentation to the committee.

Subcommittee C-3-84 — Maintenance of Way Equipment Safety.

This subcommittee was merged with the subcommittee on noise reduction on equipment, after board approval. The chairman of this subcommittee has not been able to participate as much as required in the subcommittee or committee activities and has asked to be replaced. This subcommittee must be active and a suitable replacement candidate is now being considered.

Subcommittee D-1-77 — Training Programs for Machine Operators and Maintainers

Recently the subcommittee has undertaken to update the "Handbook of Instructions for the Care and Operation of Maintenance of Way Equipment." This book was first published in 1947 and revised in 1957. It is definitely a task worth undertaking and although time consuming for committee members the end product will be appreciated by AREA members. The subcommittee is actively engaged in the standardization of training formats and with the establishment of training packages for machine operators.

Excerpts From Annual Report of A.R.E.A. Committee 28 - Clearances

J. T. Ormsby, Chairman

Subcommittee Progress

A. Recommendations for Further Study

The committee has a number of subcommittee studies which are drawing to a close. As a result, we are searching for new areas of investigation.

B. Revision of Manual

Subcommittee "B" has been looking at upgrading the quality of the graphics portion of Chapter 28 through CAD utilization. In addition, the organization and indexing of the written material is currently being looked into.

C-1-62. Compilation of the Railroad Clearance Requirements of the Various States

There have been no changes recorded by any of the states over the past year.

C-2-85. Compilation of a Comprehensive Glossary and Bibliography Pertaining to the Technical Literature on Railroad High and Wide Clearances

There have been a number of new definitions introduced by subcommittee C-2-85, as well as revisions and updates to a number of existing ones. Due to publication and revision costs, the subcommittee will hold all committee and board approved updates until such time as the number of items warrants the expenditure.

C-3-86. Liaison Committee to Coordinate with Short Line Railroads

This subcommittee has had a difficult time getting on track due to the fact that it has been necessary to chair it twice in its first two years. It may now be necessary to find yet another chairman due to Don

LaPorte's departure from the P&LE. This subcommittee's function is an essential one and we would like to continue our efforts in getting it under way.

D-1-84. Recommended Procedure to Insure that Additions or Modifications to Clearances are Promptly and Properly Reported to the Clearance Engineer

This subcommittee has been successfully brought to a close and has been submitted to the board for inclusion in the upcoming Manual.

D-2-85. Review of Railway Line Clearances to Develop Improved User Accessibility

This subcommittee may possibly be considered for ongoing status, since the Railway Line Clearance publication, as well as the railway industry itself, is constantly under change. This ongoing metamorphosis resists any attempt at assigning a realistic completion date.

D-3-85. Conversion of "Heavy Capacity and Special Type Flat Car" Section of the Official Railway Equipment Register to Umler Compatible Format

This subcommittee is currently 75% complete with 1989 as an estimated completion date.

D-4-85. Research and Develop Book Covering Heavy-Duty Car Diagrams and Ratings

The heavy-duty car bridge ratings are expected to be completed within the next year. The diagrams may take another two to three years. Overall, the subcommittee is about 50% complete.

D-5-86. Recommended Procedure for Safe Movement of Oversize and Overweight Shipments on Foreign (Jointly Operated) Track

This subcommittee has reported that it is 80% complete and will finish in 1989.

Excerpts From Annual Report of A.R.E.A. Committee 32 — Systems Engineering

R. B. Sliepka, Chairman

Subcommittee Progress:

Assignment Number	C1-1-78	C1-2-78	C2-1-86	C2-2-86	C2-3-86	C2-4-86	C3-1-78	C3-2-78	C3-3-86	C3-4-86	D4-1-86	D4-2-86
Description of Assignment	Managerial Control	Inventory Systems	Computer Applications	Data Gathering	Information Exchange	Impact of Corp. Rules	Engineer Design	Computer Graphics	Library Programs	Legal Implications	Industry Standard	Manual on Graphics
Date Assigned	1-1-78	1-1-78	1-1-86	1-1-86	1-1-86	1-1-86	1-1-78	1-1-78	1-1-86	1-1-86	1-1-86	1-1-86
Estimated % Completion	30%	75%	45%	55%	20%	10%	40%	50%	10%	10%	30%	5%
Estimated Completion Date	6-90	6-89	3-90	9-90	9-90	3-91	9-89	9-89	3-90	3-91	6-90	6-91
Problems												
Committee Priority Number												
Recommend Assignment Being Dropped/ (Discontinued)	COMBINE SUB 2	COMBINE SUB 2	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO
Recommend Assignment Being Postponed	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO

Excerpts From Annual Report of A.R.E.A. Committee 33 — Electrical Energy Utilization

A. J. Peters, Chairman

Subcommittee Assignments:

Subcommittee #1 — (C1-1-73) Electrification Economics

This assignment was made in 1973 and is an ongoing review of the economic factors influencing the installation and maintenance of railroad electrification. It is recommended that this assignment be continued at the current level of effort.

Subcommittee #4 — (C4-1-76) Railroad Electrification Systems
(D4-1-87) Electrification Safety

(C4-1-76)

This assignment was made in 1976 and consists of a review of the specification and guidelines for the major components and systems for railroad electrification, contained in Part 4 of Chapter 33 of the manual. Recently, a new section 4.1, entitled Catenary Definitions, Standards and Concepts was submitted for a manual revision. The current activity consists of the development of specifications for foundations and footings for catenary structures. A number of new techniques and designs have been used in recent years which are worthy of review. This assignment has been recommended by the committee for special emphasis during the coming year.

(D4-1-87)

This assignment was made in 1987 to review the safety requirements for the construction and operational maintenance of electrified systems. No progress has been made on this assignment to date. It is recommended that work on this task be postponed until the work on C4-1-76 is completed and a review of the assignment objectives and potential legal issues by the full committee.

Subcommittee #5 — (C5-1-76) Signals and Communications

This assignment, which was made in 1976, consists of an ongoing review of the interface requirements for railroad electrification and signal and communication equipment. There has been no significant progress in this assignment during the last year. It is recommended that this assignment be postponed pending the nomination of a new subcommittee chairman.

Subcommittee #6 — (C6-1-76) Power Supply and Distribution

This assignment was made in 1976. It consists of an ongoing review of the specifications for the power supply and distribution systems for railroad electrification. Work is progressing satisfactorily on a major update for the manual, reflecting major changes that have recently taken place in power supply technology. This assignment has also been recommended by the committee for special emphasis during the coming year.

Subcommittee #8 — (D8-1-81) Equipment Generated Electrical Noise

This assignment was initiated in 1981 to look at the impact of electrical noise generated by railroad electrification equipment on other railroad equipment and determine whether a new section in Chapter 33 was required. This assignment has received low priority to date. It is recommended that the new chairman be asked to review the current status and report back to the full committee during the coming year.

Excerpts From Annual Report Of A.R.E.A. Committee 34 — Scales

C. T. Picton, Chairman

Subcommittee Activities:

Subcommittee A — Recommendations For Further Study and Research

Recommendations For Further Study and Research is an ongoing committee.

They investigate all new innovations connected to Railroad Weighing and determine if they comply with the A.A.R. Handbook.

The Kaman Corp. has installed a new coupled-in-motion axle scale in the Kansas City, Kansas area for the purpose of testing this type system.

Subcommittee B — Revision of the A.A.R. Scale Handbook

Revision of A.A.R. Scale Handbook is an ongoing committee.

This subcommittee will remain one of the most important because Federal and State Laws must be carefully monitored for changes so the handbook can be quickly supplemented to reflect these changes. Also, innovations in scales and weighing are taking place at a faster pace and it is necessary that the handbook reflect any changes needed to accommodate the new systems. New additions to the handbook were recently published and sent to all our members.

Subcommittee C-1-85 — Preparation of Subjects For Publication

Preparation of Subjects For Publication is an ongoing committee.

Subcommittee C-2-82 — Innovations in Scales Used In Connection With Operations of Railroads

Innovations In Scales Used In Connection With Operations Of Railroads is an ongoing committee that investigates all new design and types of scales used in railroading.

Recently, the manufacturers of coupled-in-motion scales have modified their equipment so increased weighing speed could be used while keeping the scales within the tolerance allowed.

Subcommittee D-1-83 — Criteria For The Location Of Coupled-In-Motion Track Scales

The efforts of this subcommittee are directed towards writing a set of procedures to be followed by Railroads or others who are contemplating the installation of a coupled-in-motion scale.

This subcommittee will use some data that was collected for the National Bureau of Standards Advisory Group.

Subcommittee D-2-87 — Investigate Stenciling Of Cars Using Coupled-In-Motion Weights

Data for this subcommittee has been delayed due to the tragic accident that claimed our subcommittee chairman's life in December 1987. A new subcommittee chairman has been named and progress should be made with this report soon.

Proposed Manual Revisions

The following proposed Revisions of the A.R.E.A. *Manual for Railway Engineering* have been recommended to the association by the technical committee responsible for each chapter after a letter ballot is approved by: (1) a two-thirds majority of the eligible members voting, and (2) by at least fifty percent of the total eligible voting members on the committee. They are being published here for comment of the general A.R.E.A. membership and any other interested parties. Comments should be sent to A.R.E.A. headquarters by March 1, 1989. These comments will be considered by the A.R.E.A. Board of Direction in deciding whether to give final approval for inclusion of the proposed changes in the Manual Revisions which go into effect August 1, 1989.

Proposed 1989 Manual Revisions To Chapter 1 — Roadway and Ballast

The following Parts of Chapter 1 are proposed to be revised as follows:

PART 4—CULVERTS

Proposed Part 4 changes involve a total reorganizing and renumbering. A new Section 4.8, Hydraulics of Culverts, is proposed to be added. Section 4.8 was proposed as a 1988 Manual Revision and has now incorporated changes recommended to the 1988 version. (A copy of Section 4.8 is not being printed here, but is available by writing A.R.E.A. Headquarters and enclosing $2.00.)

PART 10—GEOSYNTHETICS

Proposed changes include revision of Article 10.1.2, Table 10.1.2.2, Articles 10.1.3 through 10.1.5 and the addition of new Sections 10.2 — Geotextile Specifications for Railroad Drainage Applications and 10.3 — Geotextile Specifications for Railroad Erosion Control Applications.

Revisions to the Articles and Tables in Section 10.1 incorporate ASTM proposed labeling and identification requirements on geotextiles. Also, clarification of the specification ensures that the specifier uses "minimum" average roll values and not "typical" values.

Two new geotextile specifications have been developed to address railroad applications of geotextiles for drainage (i.e., underdrains and wall drains) and erosion protection (i.e., scour protection). These specifications are based on specifications developed by Task Force 25 for these same applications of geotextiles in highway construction. Task Force 25 is a group of geotextile industry representatives, general construction contractors, and state and federal transportation engineers established to provide objective specification guidelines to end users (specifically State D.O.T.'s). The Specification Guides developed by the Task Force are meant to serve as minimum guidelines.

With this in mind, the AREA Specifications for Drainage and Erosion Control have incorporated physical property values relating to construction survivability and detailed test relating to geotextile installation from the Task Force guidelines. Additionally, these AREA specifications include filtration criteria to provide the railroad engineer with guidance in determining appropriate geotextile hydraulic properties. The object is to specify a geotextile that will survive construction and have a higher permeability than the soil, and that will not blind, clog, or pass excessive fines.

The revisions are as follows:

10.1.2 Material Requirements

The geotextile shall be a nonwoven fabric or equivalent, including composites, consisting of long chain polymeric filaments, yarns, staple fibers or other structural components of polyester, polypropylene, etc. formed into a stable network.

The geotextile shall be inert to commonly encountered chemicals, hydrocarbons, and mildew. The geotextile shall be resistant to ultraviolet light, rot, insects, rodents and conform to the properties in the following tables entitled "Geotextile Classification" and "Geotextile Requirements". The geotextile shall be tinted or otherwise treated to prevent the occurrence of snowblindness of handling personnel. The geotextile shall be resistant to abrasion from the movement of adjacent roadbed materials and of ballast. In some fabrics, resin treatment may increase abrasion resistance. The minimum average roll value (weakest principle direction) for physical properties of any roll tested from the manufacturing lot or lots or particular shipment shall be in excess of the minimum average roll value (weakest principle direction) stipulated in Table 10.1.2.2.

TABLE 10.1.2.2
PHYSICAL PROPERTY REQUIREMENTS
FOR RAILROAD TRACK STABILIZATION

These index properties are **Minimum Average Roll Values**

Test Methods for Geotextiles	Regular Typical Weight* 10-12 Oz/Sq Yd	Heavy Typical Weight* 12-16 Oz/Sq Yd	Extra Heavy Typical Weight* 16-20 Oz/Sq Yd
Grab Tensile Strength - ASTM D-4632 (lbs)	175	225	350
Elongation at Failure - ASTM D-4632 (%)	20	20	20
Mullen Burst Strength - ASTM D-3786 (psi)	400	450	620
Planar Water Flow/Transmissivity-ASTM D-4716 (Sq. Ft/min. x 10^{-3} @ Normal Stress of 3.5 psi and i = 1.0)	2	4	6
Coefficient of Normal Permeability (K) (cm/sec) ASTM D-4491	.1	.1	.1
Permittivity ASTM D-4491 ($Sec.^{-1}$)	.30	.25	.20
Apparent Opening Size ASTM D-4751 (U.S. Standard Sieve No.) U.S. Standard Sieve Number larger than	70	70	70
Trapezoid Tear Strength - ASTM D-4533 (lbs)	100	130	150
Puncture Strength - ASTM D-3787 (lbs) using 5/16" flat tip end)	110	150	185
Abrasion Resistance (No satisfactory test is available at this time; ASTM is currently working on a testing procedure.)	Not Available	Not Available	Not Available

*Fabric weight: The weights indicated for the classification of material are for information only. It is recommended that the selection of material be based on the specifications using these tables as minimum values. Material selection should not be limited by the weights shown; i.e., geotextiles may accomplish the same purposes with more or less weight.

10.1.3 Packing and Identification Requirements

The geotextile shall be provided in rolls wrapped with protective covering. A tag or other method of identification shall be attached to each wrapped roll of fabric indicating the following:

1. Manufacturer or product name
2. Date of manufacture of the fabric
3. Roll identification number
4. Customer order number (on Bill of Lading)
5. Weight per square yard of fabric (or style of fabric)
6. Width of roll
7. Length of roll

Additionally, a label shall be attached to the inside of the roll core indicating the style and roll identification number.

Each roll of fabric shall be wrapped individually and the protective covering shall be adequate for long or short term protection against moisture and ultraviolet exposure prior to placing.

10.1.4 Compliance, Inspection, and Sampling Requirements

A competent laboratory must be maintained by the producer of the geotextile, at the point of manufacture, to insure that quality control is in accordance with ASTM testing procedures. That laboratory shall maintain records of its quality control results and provide, upon request of the specifying agent prior to shipment or at any other reasonable time thereafter, a manufacturer's certificate. The certification shall be based on minimum average roll values, and shall include:

(a) Name of manufacturer

(b) Chemical composition

(c) Product description

(d) Statement of compliance to specification requirements

(e) Laboratory test results from the lots corresponding to the rolls shipped

(f) Signature of legally authorized official attesting to the information required

(g) Purchaser

It is recommended that a fabric segment be taken from every 20th roll, or at an interval specified by the Engineer in the field, and be tested at an independent competent laboratory to indicate compliance with these specifications.

10.1.5 Construction Details and Methods

The geotextile shall be installed in accordance with the plans, specifications, or as directed.

The geotextile may be spliced by overlap, sealing, or sewing. The minimum overlap distance in the transverse of longitudinal direction shall be 2 feet. Sealed or sewed seams will be allowed if the overlap in the transverse or longitudinal directions is a minimum of 6 inches, and the sealed or sewed seam grab tensile strength is equal to the minimum average roll value of the geotextile (shown on Table 10.1.2.2) in the weakest principle direction. The contractor will be allowed to patch small rips or tears in the fabric when approved by the engineer, and in accordance with the foregoing requirements.

Prior to geotextile placement, the subgrade shall be prepared and shaped to the line and grades as directed. Special attention shall be given to eliminate any sag pockets in the roadbed and to facilitate drainage away from the track centerline.

10.2 GEOTEXTILE SPECIFICATIONS FOR RAILROAD DRAINAGE

10.2.1 Introduction

10.2.1.1 Significance and Use

The use of geotextiles for drainage applications is dependent on environmental and subgrade conditions. Geotextiles shall be used, when necessary, to provide for adequate filtration and/or separation of the drainage aggregate and the subgrade soils.

Geotextiles and related products also shall have numerous other civil engineering applications apart from the application described herein. Examples of these include subgrade stabilization, ballast/subgrade separation, embankment construction over weak soils, access road stabilization, retaining wall construction, slope reinforcement, and slope erosion protection.

10.2.1.2 Application

This work shall consist of furnishing and placing a geotextile for the following drainage applications: edge drains, interceptor drains, wall drains, recharge basins, and relief wells. The geotextile shall be designed to allow passage of water while retaining insitu soil without clogging, in accordance with Table 10.2.2, and shall satisfy the construction survivability requirements of Table 10.2.1.

10.2.2 Material Requirments

Fibers used in the manufacture of geotextiles, and the threads used in joining geotextiles by sewing, shall consist of long chain synthetic polymers composed of at least 85% by weight polyolefins, polyesters, or polyamides. They shall be formed into a network such that the filaments or yarns retain dimensional stability relative to each other, including selvages.

The geotextile shall be inert to commonly encountered chemicals, hydrocarbons, and mildew. The geotextile shall be resistant to ultraviolet light, rot, insects, or rodents, and shall conform to the properties in Table 10.2.1, "Geotextile Survivability Properties", and Table 10.2.2, "Geotextile Filtration Properties". (Note: The goal or object is to specify a geotextile that will survive construction and have a higher permeability than the soil, and that will not blind, clog, or pass excessive fines.) The minimum average roll value (weakest principle direction) for properties of any roll tested from the manufacturing lot or lots, or particular shipment shall be in excess of the minimum average roll value (weakest principle direction) stipulated in Tables 10.2.1 and 10.2.2.

10.2.3 Packing and Identification Requirements

The goetextile shall be provided in rolls wrapped with protective covering. A tag or other method of identification shall be attached to each wrapped roll of fabric indicating the following:

1. Manufacturer or product name
2. Date of manufacture of the fabric
3. Roll identification number
4. Customer order number (on Bill of Lading)
5. Weight per square yard of fabric (or style of fabric)
6. Width of roll
7. Length of roll

Additionally, a label shall be attached to the inside of the roll core indicating the style and roll identification number.

Each roll of fabric shall be wrapped individually and the protective covering shall be adequate for both short and long-term protection against moisture and ultraviolet exposure prior to placing.

10.2.4 Compliance, Inspection, and Sampling Requirements

A competent laboratory must be maintained by the producer of the geotextile, at the point of manufacture, to insure that quality control is in accordance with ASTM testing procedures. That laboratory shall maintain records of its quality control results and provide upon request of the specifying agent prior to shipment or at any other reasonable time thereafter, a manufacturer's certification. The certification shall be based on minimum average roll values, and shall include:

1. Name of manufacturer
2. Chemical composition

3. Product description
4. Statement of compliance to specified requirements
5. Laboratory test results from the lots corresponding to the rolls shipped
6. Signature of legally authorized official attesting to the information required
7. Purchaser

It is recommended that a fabric segment be taken from every 20th roll, or at an interval specified by the engineer in the field, and be tested at an independent competent laboratory to indicate compliance with these specifications.

10.2.5 Construction Details and Methods

Prior to geotextile placement, the excavation shall be shaped to the lines and grades as directed. The excavation should be cleared of all large stones, roots, or debris.

Depressions or holes should be filled to permit the geotextile to be placed in intimate contact with the prepared surface.

The geotextile shall be installed in accordance with the plans and specifications, or as directed.

The geotextile shall be placed without any folds or excessive wrinkles (both longitudinally and transversely), and in intimate contact will all soil surfaces. On vertical surfaces geotextile pins may be used to secure the geotextile in place. Special care shall be taken to avoid damaging the geotextile during placement of the aggregate. The geotextile should be covered the same day as it is placed, if possible. In any event, no geotextile materials shall be exposed to ultraviolet light for a period of more than 7 days, in order to minimize any strength loss which may occur in some geotextiles. The protective shipping cover must not be removed from any rolls of geotextile until the day that it is to be installed.

In trenches, after placing the backfill material, the geotextile shall be folded over the top of the filter material to produce a minimum overlap of 12 inches for trenches greater than 12 inches wide. In trenches less than 12 inches in width, the overlap shall be equal to the width of the trench. The geotextile shall then be covered with the subsequent course.

Successive sheets of geotextiles shall be overlapped a minimum of 12 inches in the direction of flow.

Where seams are required in the longitudinal trench direction, they shall be joined by either sewing or overlapping.

Overlapped seams shall have a minimum overlap equal to the width of the trench.

The contractor will be allowed to patch small tears in the geotextile. A geotextile patch shall be placed over the damaged area and extend 12 inches beyond the perimeter of the tear or damage.

10.2.6 Measurement and Payment

The quantities of drainage geotextiles as shown on the plans may be increased or decreased at the direction of the Engineer based on construction procedures and actual site conditions that occur during construction of the project. Such variations in quantity will not be considered as alterations in the details of construction or a change in the character of the work.

The geotextile shall be measured for payment by the square yard based on the quantities shown on the plans, including overlaps, unless otherwise prescribed.

TABLE 10.2.1
GEOTEXTILE SURVIVABILITY PROPERTIES[1,2]
FOR DRAINAGE GEOTEXTILES

Property	DRAINAGE[3] Class A[4]	Class B[5]	Test Method
Grab Strength Lbs	180	80	ASTM D-4632
Elongation (%)	N/A	N/A	ASTM D-4632
Sewn Seam Strength[6] Lbs	160	70	ASTM D-4632
Puncture Strength Lbs	80	25	ASTM D-3787 (5/16" hemispherical tip)
Burst Strength psi	290	130	ASTM D-3786
Trapezoid Tear Lbs	50	25	ASTM D-4533
Ultraviolet Degradation at 150 hours	70% Strength retained for all classes		ASTM D-4355

1. Acceptance of geotextile material shall be based on ASTM D4759 "Standard Practice for Determining the Specification Conformance of Geosynthetics".
2. Contracting agency may require a letter from the supplier certifying that its geotextile meets specification requirements.
3. Minimum. Use value in weaker principle direction. All numerical values represent minimum average roll value (i.e., test results from any sampled roll in a lot shall meet or exceed the minimum values in the Table). Stated values are for non-critical, non-severe applications. Lots sampled according to ASTM D-4354.
4. Class A Drainage applications for fabrics are where installation stresses are more severe than Class B applications; i.e., very coarse, sharp angular aggregate is used, a heavy degree of compaction (95% AASHTO T99) is specified or depth of trench is greater than 10 feet.
5. Class B Drainage applications are those where fabric is used with smooth graded surfaces having no sharp angular projections, no sharp angular aggregate is used; compaction requirements are light (< 95% AASHTO T99), and trenches are less than 10 feet in depth.
6. Values apply to both field and manufactured seams.

TABLE 10.2.2
GEOTEXTILE FILTRATION PROPERTIES*
FOR DRAINAGE APPLICATIONS

Criteria	Property	M.A.R.V.**	Test Method
I. SOIL RETENTION	Apparent Opening Size (AOS), U.S. Sieve No.	70 ($\leq$.210mm)	ASTM D4751
II. PERMEABILITY	Permeability (K), cm/s	.1	ASTM D4491
III. CLOGGING	Percent Open Area	4%	CWO2215
	Porosity	30%	$\frac{\text{Volume Voids}}{\text{Total Volume}}$

*To be used when soil data is not available for non-severe/non-critical applications. For all other applications, use Table 10.2.2a.

**Minimum Average Roll Value.

TABLE 10.2.2a
GEOTEXTILE FILTRATION CRITERIA
FOR DRAINAGE APPLICATIONS

I. SOIL RETENTION (PIPING RESISTANCE CRITERIA)[1]

Soils	Steady State Flow	Dynamic, Pulsating, And Cyclic Flow
$\leq$50% Passing[2] U.S. No. 200 Sieve	AOS-- $O_{95} \leq 3\ D_{85}$	$O_{95} \leq D_{15}$ (If soil can move beneath fabric)
		or
	$C_u \leq 2$ or ≥ 8 $B = 1$	$O_{50} \leq 0.5\ D_{85}$
	$2 \leq C_u \leq 4$ $B = 0.5\ C_u$	
	$4 \leq C_u \leq 8$ $B = \frac{8}{C_u}$	
$\geq$50% Passing U.S. No. 200 Sieve	Woven: $O_{95} \leq D_{85}$	$O_{50} \leq 0.5\ D_{85}$
	Nonwoven: $O_{95} \leq 1.8\ D_{85}$	
	AOS No. (fabric) $\geq$ No. 50 Sieve	

1. When the protected soil contains particles from 1 inch size to those passing the U.S. No. 200 sieve, use only the gradation of soil passing the U.S. No. 4 sieve in selecting the fabric.
2. Select fabric on the basis of largest opening value required (smallest U.S. Sieve number).
3. Coefficient of uniformity: $C_u = d_{60}/d_{10}$
4. Apparent Opening Size: The Apparent Opening Size (AOS) of a geotextile, represented by the symbol O_{95} and with units of mm (in.), is the dimension of the largest openings in the fabric which describes its capability for retaining soil particles on the surface of the fabric (ASTM D4751).

5. D_n: D = diameter of soil particle, mm.
   n = percentage of soil, by weight, finer than the diameter, D, as determined by ASTM D422, "Standard Method of Particle-Size Analysis of Soils".

II. PERMEABILITY CRITERIA[1,2]

A. Critical/Severe Applications

k (fabric) $\geq$ 10 k (soil)

B. Less Critical/Less Severe Applications (with Clean Medium to Coarse Sands and Gravels)

k (fabric) $\geq$ k (soil)

1. Permeability should be based on the actual fabric open area available for flow. For example, if 50% of fabric area is to be covered by flat concrete blocks, the effective flow area is reduced by 50%.
2. Permittivity of a Geotextile: The Permittivity of a geotextile, represented by the symbol ψ, and with units of s^{-1} (sec^{-1}), is the volumetric flow rate of water per unit surface area, per unit head under laminar flow conditions, in the normal direction through a geotextile (ASTM D4491).
3. Permeability - The Coefficient of Normal Permeability of a geotextile, represented by the symbol k, and with units of m/s (cm/sec), is the rate of discharge of water under laminar flow conditions, through a unit surface area of a geotextile (i.e., normal to the plane of the geotextile), under a unit hydraulic gradient, and a standard water temperature of 20°C. A nominal Coefficient of Normal Permeability may be determined by multiplying the permittivity value by nominal thickness.

III. CLOGGING CRITERIA

A. Critical/Severe Applications

Select fabrics meeting I, II, IIIB, and perform soil/fabric filtration tests before specification, prequalifying the fabric, or after selection before bid closing. Alternative: use approved list specification for filtration applications. Suggested performance test method: Gradient Ratio ≤ 3.

B. Less Critical/Non-Severe Applications

1. Whenever possible, fabric with maximum opening size possible (lowest U.S. Sieve No.) from retention criteria should be specified.
2. Effective Open Area Qualifiers[2]: Woven fabrics: Percent Open Area: $\geq 4\%$.
   Nonwoven fabrics: Porosity[3] $\geq$ 30%
3. Additional Qualifier (Optional): $O_{95} \geq 3D_{15}$
4. Additional Qualifier (Optional): $O_{15} \geq 3D_{15}$

NOTE: 1. Filtration tests are performance tests and cannot be performed by the manufacturer as they depend on specific soil and design conditions. Tests to be performed by specifying agency or his representative. Note: experience required to obtain reproducible results in gradient ratio test.

2. Qualifiers in potential clogging condition situations (e.g., gap-graded soils and silty type soils) where filtration is of concern.

3. Porosity requirement based on graded granular filter porosity.

10.3 GEOTEXTILE SPECIFICATIONS FOR RAILROAD EROSION CONTROL APPLICATIONS

10.3.1 Introduction

10.3.1.1 Significance and Use

The use of geotextiles for erosion control applications is dependent on environmental and subgrade conditions. Geotextiles shall be used, when necessary, to provide for adequate *filtration* and *separation* of the armor materials and the subgrade soils.

Geotextiles and related products also shall have numerous civil engineering applications apart from the application described herein. Examples of these include subgrade stabilization, ballast/subgrade separation, embankment construction over weak soils, access road stabilization, retaining wall construction, slope reinforcement, and subsurface drainage.

10.3.1.2 Application

This work shall consist of furnishing and placing a geotextile for the following erosion control applications: cut and fill slope protection, protection of various small drainage structures and ditches, wave protection for causeways and shoreline railway embankments, and scour protection for structures such as bridge piers and abutments. The geotextile shall be designed to allow passage of water while retaining insitu soil without clogging, in accordance with Table 10.3.2, and shall satisfy the construction survivability requirements of Table 10.3.1.

10.3.2 Material Requirements

Fibers used in the manufacture of geotextiles, and the threads used in joining geotextiles by sewing, shall consist of long chain synthetic polymers composed of at least 85% by weight polyolefins, polyesters, or polyamides. They shall be formed into a network such that the filaments or yarns retain dimensional stability relative to each other, including selvages.

The geotextile shall be inert to commonly encountered chemicals, hydrocarbons, and mildew. The geotextile shall be resistant to ultraviolet light, rot, insects, or rodents, and shall conform to the properties in Table 10.3.1, "Geotextile Survivability Properties", and Table 10.3.2, "Geotextile Filtration Properties". (Note: The goal or object is to specify a geotextile that will survive construction and have a higher permeability than the soil, and that will not blind, clog, or pass excessive fines.) The minimum average roll value (weakest principle direction) for properties of any roll tested from the manufacturing lot or lots, or particular shipment shall be in excess of the minimum average roll value (weakest principle direction) stipulated in Tables 10.3.1 and 10.3.2.

10.3.3 Packing and Identification Requirements

The geotextile shall be provided in rolls wrapped with protective covering. A tag or other method of identification shall be attached to each wrapped roll of fabric indicating the following:

1. Manufacturer or product name
2. Date of manufacture of the fabric
3. Roll identification number
4. Customer order number (on Bill of Lading)
5. Weight per square yard of fabric (or style of fabric)
6. Width of roll
7. Length of roll

Additionally, a label shall be attached to the inside of the roll core indicating the style and roll identification number.

Each roll of fabric shall be wrapped individually and the protective covering shall be adequate for both short and long-term protection against moisture and ultraviolet exposure prior to placing.

10.3.4 Compliance, Inspection and Sampling Requirements

A competent laboratory must be maintained by the producer of the geotextile, at the point of manufacture, to insure that quality control is in accordance with ASTM testing procedures. That laboratory shall maintain records of its quality control results and provide upon request of the specifying agent prior to shipment or at any other reasonable time thereafter, a manufacturer's certification. The certification shall be based on minimum average roll values, and shall include:

1. Name of manufacturer
2. Chemical composition
3. Product description
4. Statement of compliance to specified requirements
5. Laboratory test results from the lots corresponding to the rolls shipped
6. Signature of legally authorized official attesting to the information required
7. Purchaser

It is recommended that a fabric segment be taken from every 20th roll, or at an interval specified by the engineer in the field, and be tested at an independent competent laboratory to indicate compliance with these specifications.

10.3.5 Construction Details and Methods

Prior to geotextile placement, the area shall be prepared and shaped to the lines and grades as directed. The area should be cleared of all large stones, roots, or debris.

Depressions or holes should either be filled, or the fabric placed to conform to the surface, in order to prevent the bridging and potential rupture of the geotextile across the hole.

The geotextile shall be installed in accordance with the plans and specifications, or as directed.

The geotextile shall be placed in intimate contact with the soil without any folds or excessive wrinkles (both longitudinally and transversely), but need not be placed in tension before placement of the riprap or other cover materials. Special care shall be taken to avoid damaging the geotextile. The geotextile should be covered the same day as it is placed, if possible. In any event, no geotextile materials shall be exposed to ultraviolet light for a period of more than 7 days, in order to minimize any strength loss which may occur in some geotextiles. The protective shipping cover must not be removed from any rolls of geotextile until the day that it is to be installed.

The geotextile may be joined by overlap, sealing, or sewing. The minimum overlap distance in the transverse of longitudinal direction shall be 2 feet, except where placed under water where the overlap shall be a minimum of 3 feet. Sealed or sewn seams will be allowed if the overlap in the transverse or longitudinal directions is a minimum of 6 inches, and the sealed or sewn grab tensile strength is equal to the minimum value of the geotextile (shown on Table 10.3.1) in the weakest principle direction. The contractor will be allowed to patch small rips or tears in the fabric when approved by the engineer, and in accordance with the foregoing requirements.

The geotextile must be firmly anchored at the top of the slope. This can most effectively be accomplished by an anchor trench, at least 3 feet from the crest of the slope, and at least 2 feet deep. The anchor trench shall be carefully compacted to ensure that good anchorage is attained. If the erosion protection is along a stream or if currents or strong water movements are expected, then the toe of the geotextile should be similarly anchored to prevent scour beneath it.

Successive geotextile sheets shall be overlapped in such a manner that the upstream sheet is placed over the downstream sheet and/or upslope over downslope. In underwater applications, the geotextile and required thickness of backfill material shall be placed the same day. The backfill placement shall begin at the toe and proceed up the slope.

Riprap and heavy stone filling shall not be dropped onto the geotextile from a height of more than one foot. Slope protection and smaller sizes of stone filling shall not be dropped onto the geotextile from a height exceeding three feet. Any geotextile damaged during placement shall be replaced as directed by the Engineer at the Contractor's expense.

Normal criteria for the sizing and selection of riprap are not superseded by the use of a geotextile filter. An intermediate layer of smaller sized stone should be placed directly onto the geotextile if very large riprap materials (e.g., > 500 pounds) are used.

10.3.6 Measurement and Payment

The quantities of erosion control geotextiles as shown on the plans may be increased or decreased at the direction of the Engineer based on construction procedures and actual site conditions that occur during the construction of the project. Such variations in quantity will not be considered as alterations in the details of construction or a change in the character of the work.

The geotextile shall be measured for payment by the square yard based on the quantities shown on the plans, including overlaps, unless otherwise prescribed.

TABLE 10.3.1
GEOTEXTILE SURVIVABILITY PROPERTIES[1,2]
FOR EROSION CONTROL APPLICATIONS

Property	EROSION CONTROL Class A[4]	Class B[5]	Test Method
Grab Strength Lbs	200	90	ASTM D-4632
Elongation (%)	15	15	ASTM D-4632
Sewn Seam Strength[6] Lbs	180	80	ASTM D-4632
Puncture Strength Lbs	80	40	ASTM D-3787
Burst Strength psi	320	140	ASTM D-3786
Trapezoid Tear Lbs	50	30	ASTM D-4533
Ultraviolet Degradation at 150 hours	70% Strength retained for all classes		ASTM D-4355

1. Acceptance of geotextile material shall be based on ASTM D4759 "Standard Practice for Determining the Specification Conformance of Geosynthetics".
2. Contracting agency may require a letter from the supplier certifying that its geotextile meets specification requirements.

3. Minimum. Use value in weaker principle direction. All numerical values represent minimum average roll value (i.e., test results from any sampled roll in a lot shall meet or exceed the minimum values in the Table). Stated values are for non-critical, non-severe applications. Lots sampled according to ASTM D-4354.
4. Class A Erosion Control applications are those where fabrics are under conditions where installation stresses are more severe than Class B applications; i.e., stone placement height should be less than 3 feet and stone weights should not exceed 250 pounds.
5. Class B Erosion Control applications are those where fabric is used in structures or under conditions where the fabric is protected by a sand cushion or by "zero drop height" placement of stone.
6. Values apply to both field and manufactured seams.

TABLE 10.3.2
GEOTEXTILE FILTRATION PROPERTIES*
FOR EROSION CONTROL APPLICATIONS

Criteria	Property	M.A.R.V.**	Test Method
I. SOIL RETENTION	Apparent Opening Size (AOS), U.S. Sieve No.	70 ($\leq$.210mm)	ASTM D4751
II. PERMEABILITY	Permeability (K), cm/s	.1	ASTM D4491
III. CLOGGING	Percent Open Area	4%	CWO2215
	Porosity	30%	$\frac{\text{Volume Voids}}{\text{Total Volume}}$

*To be used when soil data is not available for non-severe/non-critical applications. For all other applications, use Table 10.3.2a.

**Minimum Average Roll Value.

TABLE 10.3.2a
GEOTEXTILE FILTRATION CRITERIA
FOR EROSION CONTROL APPLICATIONS

I. SOIL RETENTION (PIPING RESISTANCE CRITERIA)[1]

Soils	Steady State Flow	Dynamic, Pulsating, And Cyclic Flow
≤ 50% Passing[2] U.S. No. 200 Sieve	AOS-- $O_{95} \leq 3\ D_{85}$	$O_{95} \leq D_{15}$ (If soil can move beneath fabric)
		or
	$C_u \leq 2$ or ≥ 8 $B = 1$	$O_{50} \leq 0.5\ D_{85}$
	$2 \leq C_u \leq 4$ $B = 0.5\ C_u$	
	$4 \leq C_u \leq 8$ $B = \frac{8}{C_u}$	
≥ 50% Passing U.S. No. 200 Sieve	Woven: $O_{95} \leq D_{85}$	$O_{50} \leq 0.5\ D_{85}$
	Nonwoven: $O_{95} \leq 1.8\ D_{85}$	
	AOS No. (fabric) ≥ No. 50 Sieve	

1. When the protected soil contains particles from 1 inch size to those passing the U.S. No. 200 sieve, use only the gradation of soil passing the U.S. No. 4 sieve in selecting the fabric.
2. Select fabric on the basis of largest opening value required (smallest AOS).
3. Coefficient of uniformity: $C_u = d_{60}/d_{10}$
4. Apparent Opening Size: The Apparent Opening Size (AOS) of a geotextile, represented by the symbol O_{95} and with units of mm (in.), is the dimension of the largest openings in the fabric which describes its capability for retaining soil particles on the surface of the fabric (ASTM D4751).
5. D_n: D = diameter of soil particle, mm.
   n = percentage of soil, by weight, finer than the diameter, D, as determined by ASTM D422, "Standard Method of Particle-Size Analysis of Soils".

II. PERMEABILITY CRITERIA[1,2]

A. Critical/Severe Applications

k (fabric) ≥10 k (soil)

B. Less Critical/Less Severe Applications (with Clean Medium to Coarse Sands and Gravels)

k (fabric) ≥ k (soil)

1. Permeability should be based on the actual fabric open area available for flow. For example, if 50% of fabric area is to be covered by flat concrete blocks, the effective flow area is reduced by 50%.
2. Permittivity of a Geotextile: The Permittivity of a geotextile, represented by the symbol ψ, and with units of s^{-1} (sec^{-1}), is the volumetric flow rate of water per unit surface area, per unit head under laminar flow conditions, in the normal direction through a geotextile (ASTM D4491).

3. Coefficient of Normal Permeability: The Coefficient of Normal Permeability of a geotextile, represented by the symbol k_n, and with units of m/s (cm/sec), is the rate of discharge of water under laminar flow conditions, through a unit surface area of a geotextile (i.e., normal to the plane of the geotextile), under a unit hydraulic gradient, and a standard water temperature of 20°C. A nominal Coefficient of Normal Permeability may be determined by multiplying the permittivity value by nominal thickness.

III. CLOGGING CRITERIA

A. Critical/Severe Applications

Select fabrics meeting I, II, IIIB, and perform soil/fabric filtration tests before specification, prequalifying the fabric, or after selection before bid closing. Alternative: use approved list specification for filtration applications. Suggested performance test method: Gradient Ratio ≤ 3.

B. Less Critical/Non-Severe Applications

1. Whenever possible, fabric with maximum opening size possible (lowest U.S. Sieve No.) from retention criteria should be specified.
2. Effective Open Area Qualifiers[2]: Woven fabrics: Percent Open Area: $\geq 4\%$.
   Nonwoven fabrics: Porosity[3] $\geq 30\%$
3. Additional Qualifier (Optional): $O_{95} \geq 3D_{15}$
4. Additional Qualifier (Optional): $O_{15} \geq 3D_{15}$

NOTE: 1. Filtration tests are performance tests and cannot be performed by the manufacturer as they depend on specific soil and design conditions. Tests to be performed by specifying agency or his representative. Note: experience required to obtain reproducible results in gradient ratio test.

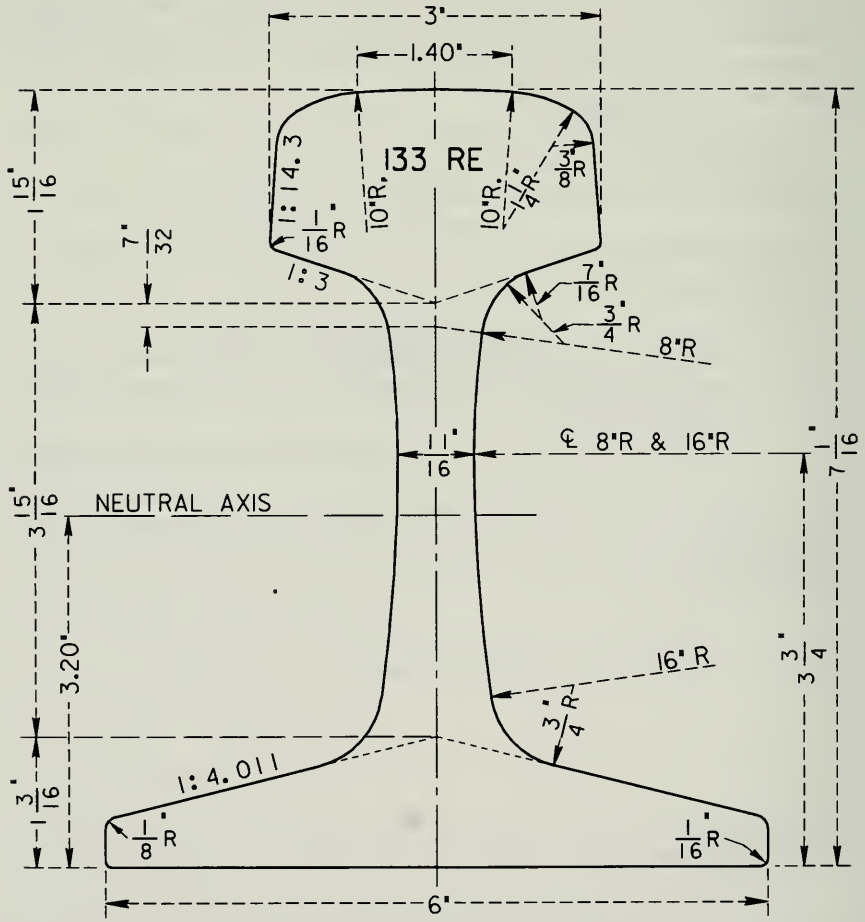
2. Qualifiers in potential clogging condition situations (e.g., gap-graded soils and silty type soils) where filtration is of concern.

3. Porosity requirement based on graded granular filter porosity.

Proposed 1989 Manual Revision To Chapter 4 — Rail

Although adopted in 1986 by the A.R.E.A. as a recommended rail section, a drawing of the 133 RE has yet to be incorporated into the Manual. The following rail plan is proposed for page 4-1-5.1.

1989
(Adopted 1986)



	Area Sq In	Percent
Head	4.85	37.1
Web	3.20	24.5
Base	5.02	38.4
TOTAL	13.07	100.0

Moment of inertia	86.2
Section modulus, head	22.3
Section modulus, base	26.9
Ratio m.i. to area	6.57
Ratio s.m. head to area	1.71
Ratio height to base	1.18
Calculated weight, lb per yd	133.4

Fig. 6 — 133 RE rail section.

Proposed 1989 Manual Revisions To Chapter 5 — Track

A standard set of specifications for insulation of track tools is proposed to be added to Part 6 — Specifications and Plans for Track Tools.

Insulation for Track Tools Standard Specifications

1989

1.0 GENERAL

1.1 Scope

This specification covers the requirements for self amalgamating sealant tape; heat shrinkable environmental sealing boots and thick wall tubing to be used for insulating track tools.

1.2 Type

(a) The products specified in this section shall be compatible with each other.

(b) Each product shall be furnished in sizes and quantities specified and manufactured in accordance with these specifications.

2.0 TECHNICAL REQUIREMENTS

2.1 Material

(a) Self-amalgamating sealant tape shall be a cross-linked Butyl based tape that fuses to itself in ambient temperatures. It shall be formulated so that it can be stretch orientated to at least four times its original length, allowing conformance to irregular shapes. The tape shall be supplied with a release paper backing, which allows at least a 1/16" overlap to avoid roll contamination and ease of release.

(b) Environmental boots shall be heat shrinkable cross linked blends of polyolefin and elastomers which provide low moisture permeability, weather resistance and resistance to ozone attack.

(c) Environmental boots shall include an adhesive system that provides an effective environmental seal that meets or exceeds the requirements of ANSI C119.1 for 600V rated systems.

(d) Heat shrinkable tubing and end caps shall be a cross-linked blend of modified polyolefin and elastomers. A sealant shall be applied inside the tubing and end caps which remains flexible for the life of the system. This sealant shall provide both environmental sealing and waterproofing. The tubing shall provide a shrink ratio of at least 3:1 and the end caps at least 2:1.

(e) The sealant system shall provide an effective environment seal to cable jacketing material, including standard plastic and elastomeric materials, yet will strip cleanly from most metallic substances, while resisting water ingress.

2.2 Properties

(a) The following properties for self-amalgamating sealant tape are requirements:

Physical Properties	Test Method	Typical Values
Water Absorption	ASTM D-570-81	0.5%
Specific Gravity	ASTM D-792-66 Method A1	1.49

Adhesion:		
Lap Shear XLPE to XLPE	ASTM D-1002-72	8-10 psi
Elongation	ASTM D-142, Method A 412-80	350-400%
Electrical Properties		
Dielectric Strength (125 mil sample)	ASTM D-149-81	250 v/mil
Volume Resistivity	ASTM D-257-78	10^{14} (min)
Dielectric Constant	ASTM D-150-81	4.9
Dissipation Factor	ASTM D-150-81	0.06
Thermal Properties		
Maximum Service Temperature	C-792-75	265°F
Low Temp. Flexibility	ASTM-3111-76	-40°F on 1/2″ mandrel
Chemical Properties		
Corrosivity	ASTM D-2671-80	Non-corrosive 16 hrs. @ 150°C
Chemical Resistance	ASTM D-2671-80	Resistant to: alcohol, ketones, acids, alkalis. Non-resistant to: Mineral spirits, naptha, hydrocarbons.
Fungus Resistance	ASTM G-21	Rating 0

(b) The following properties for environmental boots are required:

Physical Properties	**Test Method**	**Results**
Tensile Strength, psi	ASTM D-412 Method A	1500 (min)
Ultimate Elongation (%)	ASTM D-412 Method A	440 (min)
Hardness - Shore "D"	ASTM D-2240	55 ± 5
Water Absorption (%)	ASTM D-570 24 hours	0.1% (max)
Specific Gravity	ASTM D-792 Method A1	1.22 ± .12
Electrical Properties		
Dielectric Strength 125 mil 125 mils (v/mil)	ASTM D-149	350 (min)
Dielectric Constant (1 kHz)	ASTM D-150	4.5
Volume Resistivity (ohm-cm)	ASTM D-257	10^{14} (min)

(c) The following properties for heat shrinkable tubing and end caps must be adhered to:

Physical Properties	**Test Method**	**Results**
Tensile Strength, psi	ASTM D-412 Method A	1800 (min)
Ultimate Elongation (%)	ASTM D-412 Method A	400% (min)
Hardness - Shore "D"	ASTM D-2240	48 ± 5
Water Absorption (%)	ASTM D-570	0.1% (max)
Specific Gravity	ASTM D-792 Method A1	1.08 ± .08

Chemical Properties		
Corrosivity	ASTM D-2671	Non-corrosive
	Method A	16 hrs. @ 175°C
Fungus Resistance	ASTM G-21	Rating 0
Oil Resistance - 24 hrs. @ 25°C	ASTM D-412	Rating 0
Hydraulic Fluid Mil H5606 °C	% Original Strength	90 (min)
	% Original Elongation	100 (min)
Diesel Fuel	% Original Strength	100 (min)
	% Original Elongation	120 (min)
Electrical Properties		
Dielectric Strength 125 mils (v/mil)	ASTM D-149	350 (min)
Dielectric Constant (1 khz)	ASTM D-150	3.4 (min)
Volume Resistivity (ohm-cm)	ASTM D-257	10^{14} (min)
Dissipation Factor (1 khz)	ASTM D-150	0.005 nominal

2.3 Sizes

(a) The following is a size selection chart for self amalgamating sealant tape. The size, length and quantity manufactured shall be as specified:

Width (Inches)	Length (Feet)	Thickness (Inches)
1	25	1/16 (min)
3-3/4	10	1/8 (min)

(b) The following is a size selection chart for environmental boots. The size, length and quantity manufactured shall be as specified:

Expanded Diameter (Inches)	Recovered Diameter (Inches)	Nominal Length (Inches)	Recovered Wall Thickness (Inches)
0.75	0.37	3.0	0.10
1.50	0.70	3.5	0.10
2.50	1.20	4.0	0.10
3.60	2.20	4.0	0.14
4.50	2.20	6.0	0.16

(c) The following is a size selection chart for heat shrinkable tubing and end caps with sealant. The size, length and quantity manufactured shall be as specified. Tubing shall be supplied up to and including 60" lengths with sealant applied. The recovered inside diameter dimensions are reduced by the flow of the sealant.

(d) End Caps

Nominal Length (Inches)	Min. Expanded ID (Inches)	Max. Rec. ID (Inches)	Nominal Recovered Wall (Inches)
3.250	1.150	0.500	0.095
3.500	1.500	0.750	0.100
3.750	2.000	1.000	0.105
4.500	2.500	1.250	0.110

Expanded Diameter (Inches)	Recovered Diameter (Inches)	Recovered Wall Thickness (Inches)
0.40	0.15	0.060
0.75	0.22	0.090
1.10	0.37	0.105
1.50	0.50	0.120
1.70	0.50	0.120
2.00	0.75	0.155
2.70	0.90	0.155
3.00	1.25	0.155
4.00	1.75	0.155
4.50	1.75	

2.4 Tolerances

Tolerances shall be as stated in the pertinent test method under the properties part of this specification.

2.5 Manufacture

(a) Tubing and end caps can be black in color for ultra violet protection. Other colors available (without ultra violet protection) are green, white and red.

(b) All insulated tools shall be clean, without burrs or rough edges and have a uniform cross section throughout and finished in a first class, workman like manner.

2.6 Drawings

The manufacturer shall submit for approval to the railroad, shop drawings of each product to be manufactured. Such drawings shall consist of scale and full size drawings showing in detail all dimensions, kind and quantity of materials, specifications and any other information required for the fabrication of each product. No work indicated by such shop drawings required for the fabrication of said item is to commence until the manufacturer has received written approval of the submitted drawings.

3.0 INSPECTION AND SHIPMENT

3.1 Inspection

The manufacturer shall have adequate facilities for the inspection of each product by a representative of the railroad during the course of manufacturing.

3.2 Testing and Acceptance

If the product offered by the manufacturer is not a standard catalog item in production for at least three (3) years the vendor will, prior to delivery, submit for approval seven (7) copies of notarized certified test reports from a recognized independent laboratory (approved by the railroad) that the material lot to be supplied is in accordance with the specifications contained herein.

3.3 Packing

Each product shall be carefully prepared for shipment to prevent any damage in transit. Any item found damaged in shipment will be rejected by the railroad and shall be removed.

4.0 INSPECTION AND USE

4.1 Tools must be inspected before each use to insure that there has been no damage to the insulation. Tools with damaged insulation must not be used.

4.2 Tools should be stored in location free of oils and grease to insure proper protection of the insulation.

Proposed 1989 Manual Revisions To Chapter 6 — Buildings

A new Part 5 pertaining to energy conservation and audits is proposed for Chapter 6. The energy conservation strategies outlined in Part 5 discuss new energy cost reduction technologies that can be applied by those charged with the responsibility to formulate reports, building energy auditing, and those responsible for maintaining and operating railway buildings and facilities. (A copy of the 59 page proposed Part 5 is not being printed here but is available by writing A.R.E.A. Headquarters and enclosing $3.00.)

Proposed 1989 Manual Revisions To Chapter 8 — Concrete Structures and Foundations

The following Parts of Chapter 8 are proposed to be revised as follows:

Part 2 — Reinforced Concrete Design:

Article 2.11.1 (b) on Longitudinal Reinforcement is proposed to be revised to:

(b) When the cross section is larger than that required by consideration of loading, a reduced effective area may be used. The reduced effective concrete area shall not be less than that which would require 1 percent of longitudinal reinforcement to carry the loading.

Part 3 — Footing Foundations, Part 5 — Retaining Walls and Abutments, Part 10 — Reinforced Concrete Culvert Pipe and Part 19 — Rules for Rating Existing Concrete Bridges:

These 4 parts have been reviewed and updated. Complete text for new Parts 3, 10 and 19 are shown on the accompanying pages 56, 67 and 80 respectively. A copy of Part 5 is available by writing A.R.E.A. headquarters and enclosing $2.00.

Part 13 — Precast Concrete Box Culverts

A new Part 13 is proposed which provides specifications for the design, manufacture and construction of precast concrete box culverts that will be supported on soil and rock foundations. The new Part 13 is shown on accompanying pages 86-89.

Part 40 — ASTM Specification References

It is proposed that Part 40 on ASTM Specification References be deleted from Chapter 8 material in the manual.

Part 17 — Prestressed Concrete Design

Revisions as follows are proposed to Articles in Part 17:

Article 17.6.4 (a) (2); Revise the last lines to read:

Members with bonded auxiliary reinforcement provided in the tensile zone to resist the total tensile force in concrete computed with the assumption of an uncracked section. $7.5\sqrt{f'_{c_i}}$

Article 17.6.5(a)(1): The current requirement 17.6.5(a)(1) will be renumbered 17.6.5(a)(1)a. Two additional requirements b. and c. are to be added to 17.6.5(a)(1) as follows:

b. Slight overstressing of pretensioning tendons up to 0.85 f_{py} for short periods of time may be permitted to offset seating losses, provided the stress after seating does not exceed the above values.

c. Overstressing of post-tensioning tendons up to 0.90 f_{py} for short periods of time may be permitted to offset seating and friction losses provided the stress at the anchorage does not exceed the above value. The stress at the end of the seating loss zone must not exceed 0.82 f_{py} immediately after seating.

Article 17.6.5(a)(2); The current requirement will be rewritten to:

a. Stress-relieved pretensioning tendons immediately after prestress transfer . . . 0.82f_{py} or 0.70 f_{pu} whichever is larger.

b. Stabilized (low-relaxation) pretensioning tendons immediately after prestress transfer . . . 0.82f_{py} or 0.75 f_{pu} whichever is larger.

Article 17.6.6(d); Revise the last paragraph to read:

Data representing properties and effects of lightweight concrete shall be determined by tests or supplied by the manufacturer.

Article 17.6.6(d)(1); Revise last sentence in definition of f_{cr} to read:

Amount of reduction below initial prestress can be estimated, or for pretensioned members the reduced tendon stress may be taken as 0.63f_{pu}, for stress relieved strand or 0.69 f_{pu} for low relaxation strand.

Article 17.6.6(d)(4); Revise to read as follows:

(4) Relaxation of tendon stress

For Pretensioning tendons:

a. (250 or 270 ksi stress-relieved strand tensioned to 0.70 f_{pu})

$$f_r = 20{,}000 - 0.4\, f_e - 0.2\,(f_s + f_c) \qquad (17\text{-}20a)$$

b. (270 ksi low-relaxation strand tensioned to 0.75 f_{pu})

$$f_r = 5000 - 0.10\, f_e - 0.05\,(f_s + f_c) \qquad (17\text{-}20b)$$

For Post-tensioning tendons:

a. (250 or 270 ksi stress relieved strand anchored at 0.70 f_{pu})

$$f_r = 20{,}000 - 0.3\, f_f - 0.4\, f_e - 0.2\,(f_s + f_c) \qquad (17\text{-}21a)$$

b. (270 ksi low-relaxation strand anchored at 0.75 f_{pu})

$$f_r = 5000 - 0.07\, f_f - 0.1\, f_e - 0.05\,(f_s + f_c) \qquad (17\text{-}21b)$$

c. (240 ksi stress-relieved wire)

$$f_r = 18{,}000 - 0.3\, f_f - 0.4\, f_e - 0.2\,(f_s + f_c) \qquad (17\text{-}22)$$

d. (145 to 160 ksi high-strength steel bar)

$$f_r = 3{,}000$$

where:

f_f = friction loss below 0.70 f_{pu} at point being considered, computed according to paragraph (d)(6).

f_e, f_s, f_c = appropriate values as determined for either pretensioned or post-tensioned member.

Article 17.6.6(d)(6)(e); Revise the table at the end of this paragraph to read:

Type of Prestressing tendon	Total loss of prestress	
	f'_c = 4,000	f'_c = 5,000
Pretensioning strand:		
Stress relieved		45,000 psi
Low relaxation		35,000 psi
Post-tensioning wire or strand:		
Stress relieved	32,000 psi	33,000 psi
Low relaxation	24,000 psi	25,000 psi
Post-tensioning bar	22,000 psi	23,000 psi

AMERICAN RAILWAY ENGINEERING ASSOCIATION

Part 3

Footing Foundations

Revised 1989

3.0 SPECIFICATIONS FOR DESIGN OF SPREAD FOOTING FOUNDATIONS

3.1 DEFINITIONS

3.1.1 General

(a) A spread footing is a structural unit which transfers and distributes a load to the underlying soil or rock at a pressure consistent with the requirements of the structure and the supporting capacity of the soil or rock. Sizing of footings on rock is not a consideration. The designer should be aware that the approaches presented here are for the least complicated situations; and where unusual geology or loadings are encountered, geotechnical engineering specialists should be consulted.

3.1.2 Classification

3.1.2.1 Spread footings may be classifed according to their structural arrangement:

(a) An individual column footing which supports a single column or isolated load.

(b) A wall footing or continuous footing which supports a wall.

(c) A combined footing which supports more than one column.

(d) A raft or mat footing, which supports all the columns in a structure or a large portion thereof.

(e) Cantilever footing which supports a wall column near its edge without causing non-uniform soil pressure.

3.1.2.2 Spread footings may be classified according to their depth and dimensions:

(a) Shallow footing for which the depth of foundation, D_f, defined as the minimum vertical distance from the base of the footing to the surface of the surrounding ground or floor, does not exceed the least width, B, of the footing. See Fig. 3.1.2.2 (a).

(b) Deep footings, for which the depth, D_f, is greater than the width, B, are described in Part 4 and Part 24 of Chapter 8 of the AREA manual.

3.1.2.3 Spread footings may be classified with respect to the subsurface material from which they derive their support:

(a) Footings on granular, non-cohesive soils.

(b) Footings on saturated clay or plastic silt.

(c) Footings on unsaturated clay or silt.

(d) Footings on non-homogeneous deposits.

(e) Footings on pre-consolidated clay.

3.2 INFORMATION REQUIRED

3.2.1 Field Survey

(a) All available information shall be furnished in the form of a topographic map, in order to adapt the structural requirements to the field conditions. The locations and dimensions of underground utilities, existing foundations, roads, tracks, or other structures shall be indicated. In connection with

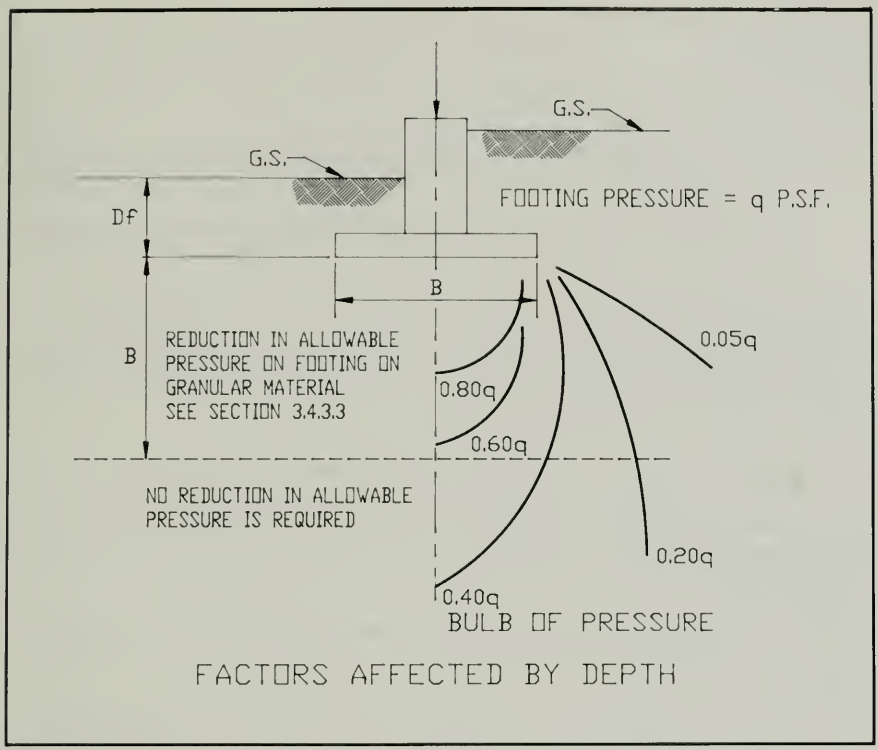


Figure 3.1.2.2 (a)

footings for river crossings, the records of normal high water, low water, flood-water level, depth of scourk, stream velocities, and alignment of the stream shall be provided.

(b) All available information concerning the nature of the foundations of neighboring structures, the nature of the underlying materials, and of the settlement and behavior of these foundations shall be assembled and condensed as a guide to the judgment of the engineer in the design of the new structure.

3.2.2 Controlling Dimensions

(a) Information shall be assembled concerning the proposed arrangement of the column, piers, abutments or equipment to be supported; the depths of basements, tunnels, and other excavations; the surface elevation of fill areas; and all other factors that may affect or be affected by the construction.

3.2.3 Loads

(a) The loads to be supported by the foundations shall be indicated. These shall be subdivided into the following categories:

1. Dead load.
2. Normal live load, defined as the live load that is likely to be transmitted to the foundation throughout the greater portion of the useful lifetime of the structure.
3. Maximum live load, defined as the greatest live load that may be anticipated at any time during the lifetime of the structure.

4. Longitudinal and lateral forces.
5. Snow load.
6. Ice load.
7. Earthquake load.
8. Wind load.
9. Loads from pore water pressures including buoyancy and seepage forces.
10. Area load, defined as any load transmitted to the supporting soil by the addition of fill or adjacent structures.
11. Impact normally is not considered in the design of a footing except for special circumstances.
12. Vibratory loads to footings on granular material shall be considered.

(b) An estimate shall be made of the duration of each loading, because the settlement of some types of subsurface materials depends upon the proportion of the total time the loads are active.

(c) The character, frequency, and amplitude of any vibratory loads including earthquakes shall be noted for further analysis. If such loads are an important consideration, the foundation design shall be referred to a geotechnical engineer with expertise in dynamics.

(d) Footings shall be designed by using the following combinations of loads:

Primary: Dead + Live + Centrifugal Force + Earth Pressure + Pore Water Pressures + Area Load + Special Vibratory Loads

Secondary: Longitudinal Force + Wind + Ice and Stream Flow Pressures + Seismic Forces

3.2.4 Character of Subsurface Materials

3.2.4.1 General

(a) Pertinent supplementary data with respect to local geological or foundation conditions, including aerial photographs and agricultural soil maps, should be assembled if available. Data concerning changes in groundwater level should also be investigated.

(b) The data concerning subsurface materials shall be assembled in suitable graphical form, such as cross sections through the various deposits, showing the probable arrangement and sequence of lenses or strata, the pertinent physical properties of each element of the deposit, and the location of the groundwater table.

3.2.4.2 Field Investigation

(a) The nature and extent of the various formations of soil and rock beneath the site and the depth to groundwater shall be determined by means of test borings or probes and physical tests of a type and to an extent appropriate to the chracter and importance of the structure and the nature of the subsurface materials. The borings shall be made in accordance with the AREA recommendations in Chapter 8, Part 22.

(b) For major structures, at least one boring should, if practicable, extend into bedrock. Borings should at least extend to a depth equal to three times the least footing width plus the depth of the footing from the ground surface. For major structures on cohesive soils undisturbed samples should generally be recovered for laboratory testing. The recovery of undisturbed samples in granular soil has not proven satisfactory. In situ tests may provide useful data for foundation design. These tests include standard penetration test, vane shear test, Dutch cone penetration test (static penetration test), pressuremeter test, and other tests as described in Chapter 8, Part 22 of this manual.

3.2.4.3 Depth of Frost and Volume Change

(a) The maximum depth of frost penetration shall be determined, usually on the basis of local experience and records. Figure 3.2.4.3 is a map showing the depths of frost penetration in the contiguous 48 states. Similarly, in regions of excessively swelling or shrinking soils, the depth to which significant volume changes occur as a result of seasonal variations in moisture content shall be determined.

(b) Permafrost, or permanently frozen ground, exists in the northern hemisphere in arctic and subarctic regions. Although the southern boundary of permafrost is irregular, it may extend as far south as the 50th parallel. Foundations for structures, in areas of permafrost, should be designed in such a way as to not disturb the permanently frozen ground; or if this is impossible, the influence of the foundation on the permafrost should be predicted so the effect of the changes can be accommodated in the design. A geotechnical engineer with experience in these ground conditions should be consulted for design of foundations to be placed on permafrost.

3.3 DEPTH OF BASE OF FOOTINGS

3.3.1 Selection of Tentative Depths

(a) On the basis of the data concerning the subsurface materials, tentative elevations for the bases of the footing should be selected. Unless special provisions are made, the depth shall not be less than the depth of frost penetration, scour, or, in expansive clay subsoils, less than the thickness of the zone of significant volume change of the subsoil due to seasonal moisture variations. Footings should be placed below disturbed shallow soils, uncontrolled fills, collapse susceptible soils, and organic soils.

3.3.2 Revision of Depths of Footings

(a) After the preliminary depths have been selected, the allowable soil pressure shall be determined and the sizes of the footings proportioned to the pressures. If the resulting design is not feasible or economical, similar studies shall be made for footings established at other depths until the most suitable and economical arrangement is determined. In considering the relative economy of footings at various levels, the cost and difficulty of excavation below groundwater level in pervious soils shall be taken into account. The economy and suitability of other types of foundations, such as piles or drilled shafts, shall also be considered. For deep foundations, the designer should refer to Part 4, Pile Foundations and Part 24, Drilled Shaft Foundations in Chapter 8 of the AREA Specifications.

3.4 SIZING OF FOOTINGS

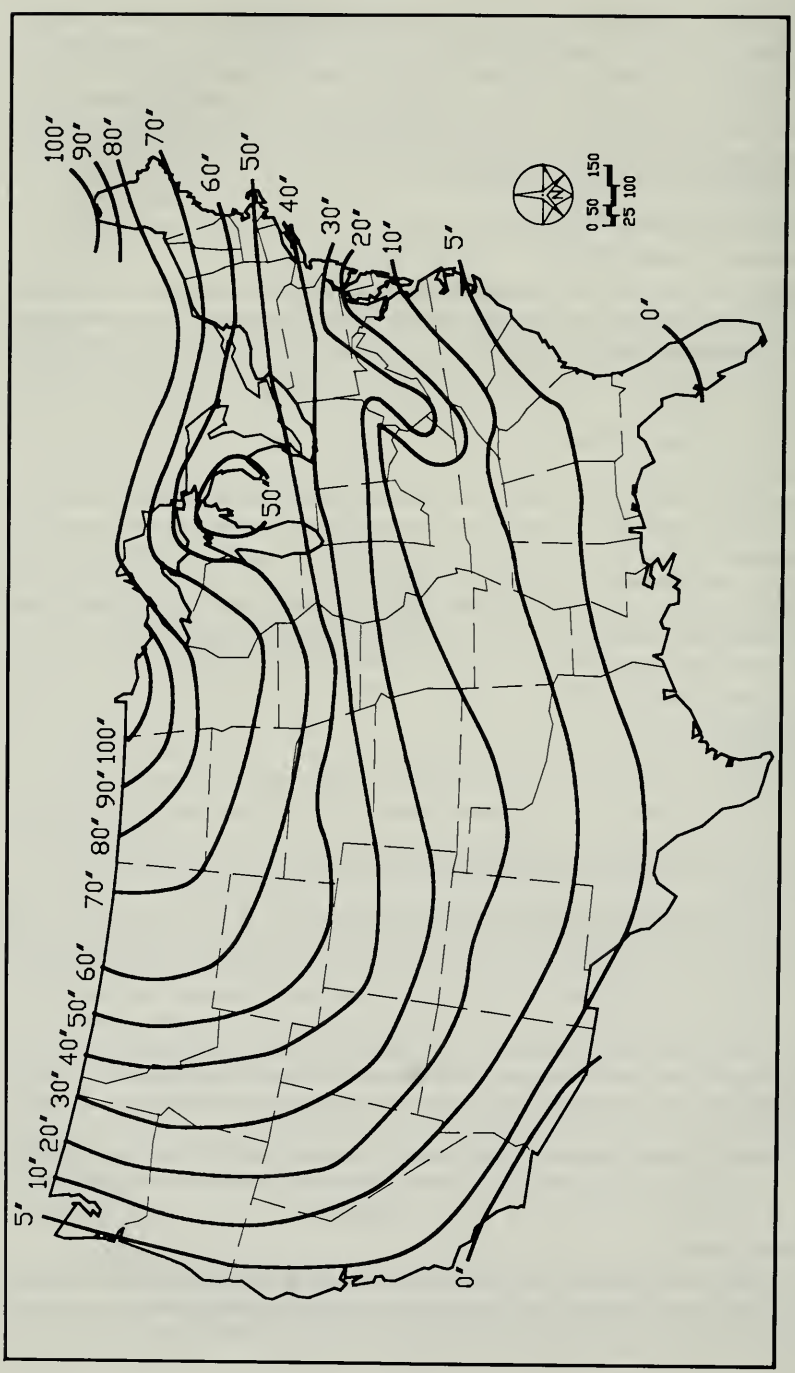
3.4.1 Definitions

(a) The following definitions will be used in the design procedures described below.

1. "Net bearing capacity," is the ultimate pressure at which the supporting material will fail in shear beneath the footing, less the pressure due to the weight of the soil at that depth.
2. "Allowable bearing capacity" is the net bearing capacity divided by an appropriate factor of safety.
3. "Allowable settlement pressure" is the maximum pressure to which the footings of the structure may be subjected without producing excessive settlement or excessive differential settlement of the structure. This settlement consists of two stages: (1) "initial settlement" or "elastic settlement" which occurs shortly after loading and (2) "consolidation" which occurs over an extended time period. The pressures for settlement are net pressures; that is, they represent pressures at the base level of the footing in excess of pressures at the same level due to the weight of the surrounding soil immediately adjacent to the footing.
4. "The allowable soil pressure" shall be taken as the smaller of the allowable bearing capacity or the allowable pressure for settlement.
5. "Contact pressure" is the total load divided by the area for vertically loaded footings and the maximum pressure applied by the combined effects of vertical and horizontal loads for eccentrically loaded footings as described in Section 3.5

3.4.2 Safety Factors

(a) The safety factor for Primary Laods shall not be less than 3; for Primary + Secondary Loads the safety factor shall not be less than 2. Additional consideration shall be taken of load duration in relation to foundation soil type and groundwater conditions when selecting a safety factor.



Extreme frost penetration, in inches, based upon state averages
Source: U.S. National Weather Records Center.
Figure 3.2.4.3

3.4.3 Shallow Footings on Granular Material (Cohesion = 0)

3.4.3.1 General

(a) The allowable soil pressure for a shallow footing on granular material depends on the width B of the footing, the shape of the footing, the depth of foundation D_f, the unit weight of the foundation material, and the position of the groundwater table.

(b) The location of the present and/or future groundwater level will noticeably affect the bearing capacity and allowable settlement pressure of the footing. Due consideration should be given to the future groundwater level.

(c) Vibrational loads can cause severe settlement of a footing founded on loose to medium granular soils. If future construction in the immediate area will require pile driving, vibratory compaction of sub-soil, or other vibrations, then consideration should be given to a more extensive vibratory analysis and a geotechnical engineer knowledgeable in soil dynamics should be consulted.

3.4.3.2 Net Bearing Capacity of a Footing on Granular Material (Cohesion = 0)

(a) The net bearing capacity of a footing on sand can be calculated from the following formulae:

For a continuous footing:

$$Q_u = 0.5 \gamma B N_\gamma + D_f \gamma (N_q - 1)$$

For a square footing:

$$Q_u = 0.4 \gamma B N_\gamma + D_f \gamma (N_q - 1)$$

For a circular footing:

$$Q_u = 0.3 \gamma B N_\gamma + D_f \gamma (N_q - 1)$$

where Q_u is the net bearing capacity in lbs./sq.ft., B is the footing width in ft., D_f is the footing depth in ft., γ is the unit weight of the sand in lb./cu. ft., and N_γ and N_q are dimensionless bearing capacity factors which are a function of Φ, the internal angle of friction, or of N, the standard penetration blow count. The standard penetration blow count is described in Chapter 8, Part 22 of the AREA Manual. The relationships between Φ, N, and the bearing capacity factors are shown in Fig. 3.4.3.2(a) as proposed by Peck, Hanson and Thornburn.

(b) For saturated sands the buoyant unit weight should be used in the equations above.

3.4.3.3 Allowable Settlement Pressure for Sand

(a) An empirical equation by Meyerhof may be used to estimate the allowable settlement pressure, Q_S, of a footing on sand. For $B \leq 4$ ft.:

$$Q_S = Ns/8$$

For $B > 4$ ft.:

$$Q_S = Ns/12 \frac{(B + 1)}{B}$$

where Q_S is in tons/sq.ft., N is the standard penetration blow count, B is the footing width in ft. and s is the allowable settlement in inches. The presence of a water table will have the effect of reducing the allowable settlement pressure as the effective stress is lowered. Therefore the allowable settlement pressure shall be reduced 50% if the water table is at the base of the footing and 0% if the water table is at a depth greater than B. The reduction for intermediate depths can be interpolated.

3.4.3.4 Sizing Footings on Granular Material

(a) Once the net bearing capacity has been determined from 3.4.3.2, the allowable bearing capacity can be determined by dividing by the appropriate safety factor as described in articles 3.4.1(a)2 and 3.4.2.

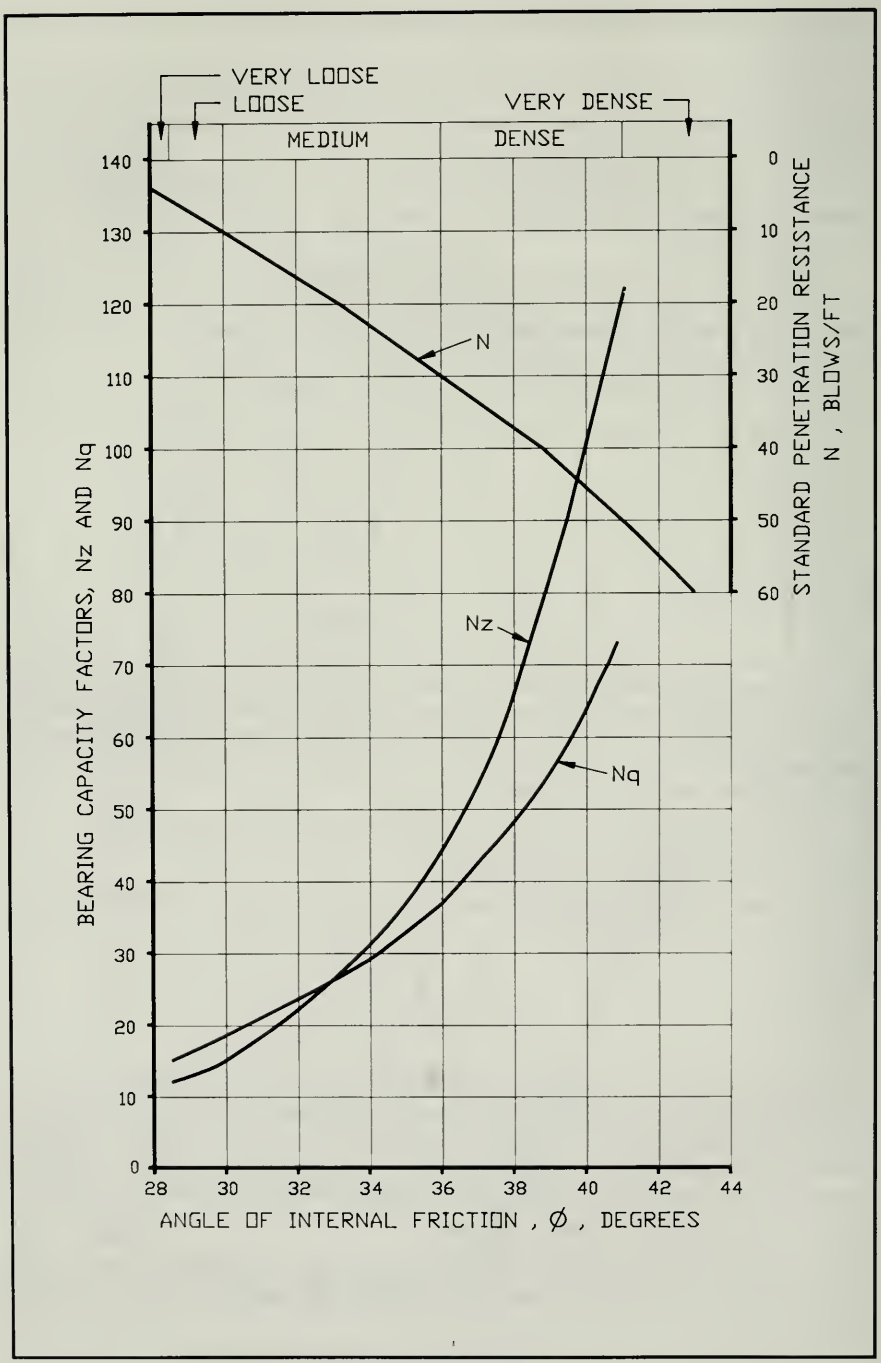


Figure 3.4.3.2 (a)

3.4.4 Shallow Footings on Saturated Clay ($\phi = 0$)

3.4.4.1 Net Bearing Capacity (Q_u)

(a) The net bearing capacity of shallow footings on saturated clays or clayey soils depends on the footing width, B; the footing length, L; the depth, D_f, of the footing below the surface and on the unconfined compressive strength, q_u, of the clay. The net bearing capacity for a footing may be determined by means of the following equations.

For a continuous footing:

$$Q_u = 2.7\ q_u$$

For a square or rectangular footing:

$$Q_u + 2.7\ q_u\ (1 + 0.3\ B/L)$$

For a circular footing:

$$Q_u = 3.5\ q_u$$

(b) In these equations, Q_u and q_u are expressed in tons/sq. ft. The value of q_u shall be taken as the average unconfined compressive strength of the clay within a depth B below the base of the footing; provided, however, that the strength of the clay does not decrease significantly with increasing depth below the footing. In the event that soft material underlies stiffer material, a special investigation of the bearing capacity of this level shall be undertaken.

3.4.4.2 Sizing Footings on Clay

(a) The correct factor of safety as indicated in articles 3.4.1.(a)2 and 3.4.2 shall be used in order to obtain an allowable bearing capacity. The required footing area is determined by dividing the column or wall load by the allowable bearing capacity.

3.4.4.3 Settlement Characteristics

(a) For footings located on or above medium clays, (q_u below 2.0 tons per sq. ft.) settlement analysis should generally be undertaken using the footing size and contact pressure determined in article 3.4.1.(a)5. In certain cases, large settlements will occur by consolidation of an underlying layer under very small additional loads. If any doubt exists concerning the consolidation characteristics of the soil, one or more consolidation tests should be undertaken. Settlement by "consolidation" of underlying clay layers can be many times the initial "elastic settlement." Both the consolidation and elastic settlements can be estimated by laboratory analysis. If the estimated settlement is greater than the allowable settlement, the footing size shall be increased to bring the estimated settlement below the allowable limit or a deep foundation shall be used.

(b) The effect of subsidance due to drainage of the soil shall be considered in the design of the structure.

3.4.5 Footings on Unsaturated Silts and Clays

(a) Accurate determination of the bearing capacity of such soils is very difficult; and complicated laboratory testing is required. Due to the existence of hairline cracks in the soil structure, and unknown pore-air pressures, an extensive field investigation may be required. Each structure will have a different solution. Careful evaluation is necessary in order to arrive at a satisfactory footing design. A rise in the groundwater table will reduce the allowable capacity and complicate the analysis.

(b) Where loadings on footings are light, or in the case of a floor slab, roadway, walks or other similar lightly loaded areas, due consideration to swelling of a clay soil shall be given. This may be especially important if the percent of soil with particle diameters less than 0.001 mm is greater than 15%.

3.4.6 Footings on Non-Homogeneous Deposits

(a) Footings established above stratified or other non-homogeneous formations shall be proportioned on the assumption that the most unfavorable condition disclosed by the subsurface exploration may be present under the most heavily loaded footings, unless detailed information is obtained concerning the actual conditions beneath each footing.

(b) Subsoil of this type requires extensive knowledge and investigation in order to obtain an accurate solution. However, in many cases using the above assumption in order to simplify the solution is satisfactory.

3.4.7 Footings on Soils with Cohesion and Friction (Preconsolidated Clays)

(a) Many soils fit this category and an accurate analysis can be carried out. The investigation must be undertaken without the use of the simplifying assumptions made for granular or cohesive soils, and more extensive laboratory information is required. Triaxial shear tests are required for this analysis.

(b) At times, it will be satisfactory to assume the soil alternately only granular or cohesive and use the lower value for allowable pressure.

3.5 FOOTINGS WITH ECCENTRIC LOADS

3.5.1 Loads Eccentric in One Direction

(a) In cases where a footing is subjected to moments in addition to vertical loads, the line of action of the resultant force is located some distance from the centerline of the footing. This distance, called eccentricity, e, is calculated by the equation

$$e = M/P$$

where M is the moment and P is the total vertical load. The total vertical load is equal to live load plus dead load. The eccentricity shall have a maximum value of B/6.

(b) The pressure distribution beneath a footing subjected to moment will be non-uniform and the maximum pressure, P_{max} and minimum pressure, P_{min}, can be calculated from:

$$P_{max} = P/BL + 6M/B^2L$$
$$P_{min} = P/BL - 6M/B^2L$$

where B is the footing width and L is the footing length and P and M are as defined previously.

3.5.2 Loads Eccentric in Two Directions

(a) In cases where a footing is subjected to moments in two directions, the vertical load, P, is calculated by adding dead loads to live loads. Horizontal loads and their lines of action in each direction are determined, and the moments in the two directions are computed by multiplying the force times the moment arm for each load. The eccentricity in each direction is computed by dividing the moment in each direction by the vertical load as follows:

$$e_x = M_x/P \text{ and } e_y = M_y/P$$

where e_x and e_y are the eccentricities in the two directions and M_x and M_y are the moments in the respective directions. Next, select trial footing dimensions B and L, where B is the footing dimension parallel to the x direction and L is parallel to the y direction. Using these dimensions, the previously determined eccentricities, and the vertical load, calculate the maximum and minimum contact pressures beneath the footing according to:

$$P_{max} = P/BL\,(1 + 6e_x/B + 6e_y/L)$$
$$P_{min} = P/BL\,(1 - 6e_x/B - 6e_y/L)$$

where all terms are as previously defined. If P_{min} is negative, that corner of the footing is in tension and so larger footing dimensions should be tried. The computations of maximum and minimum pressures are repeated with new trial dimensions until P_{min} is positive. This indicates that the entire

footing is in compression and the entire surface area will contribute to the footing's load carrying capacity.

3.5.3 Sizing Footings with Eccentric Loads

(a) Footings shall be designed using Primary Loads with the required factor of safety and checked by using Primary + Secondary Loads with their required factor of safety. Both design criteria must be met.

(b) If the footing is subjected to eccentric loads, the maximum footing contact pressure as determined in either article 3.5.1 or 3.5.2 is compared with the allowable soil pressure determined from either articles 3.4.3.3 or 3.4.3.4 for sands, or 3.4.4.2 for clays. In the case of clays, the settlement should be estimated according to article 3.4.4.3. If the contact pressures are less than allowable pressures and the amount of settlement is acceptable, the footing size is adequate; however, if the maximum contact pressure exceeds the allowable soil pressures or if the settlement is excessive, the footing size shall be increased in order to decrease maximum contact pressure and settlement. If the resulting footing size is too large to be practical, deep foundations, such as piles as described in Part 4 or drilled shafts as described in Part 24, shall be considered.

3.6 FOOTING STRESSES

3.6.1 Loads Eccentric in Two Directions

For computation of the stress within the footing itself, the pressures on the foundation may be calculated by the procedure given in 3.5.2:

$$P = P/BL\ (1 \pm 6e_x/x \pm 6e_y/y)$$

However it is desirable, if possible, to proportion the footing for an equal pressure distribution.

A more detailed study may be required for a flexible, combined, or mat footing. In actual practice the pressure distribution may vary materially from this ideal distribution. The correct distribution of the reaction is dependent upon the rigidity of the footing, distribution of the loading, characteristics of the soil, and the factor of safety.

3.7 FIELD CONDITIONS

3.7.1 Modification of Design

If excavation discloses soils or soil conditions different from those upon which the design of the footings has been based, the design shall be altered as necessary. The plans for the footing should indicate the type of soil and soil pressure upon which the design is based.

3.7.2 Reinforcement

Wherever the concrete of a reinforced footing is in contact with the soil, steel reinforcement shall be provided with a cover of not less than 3 in. If the concrete is placed against a seal coat or against steel sheeting that is to remain in place, the cover shall be not less than 2 in.

3.7.3 Footings at Varying Levels

If the footings for two adjacent parts of a structure are established at different levels, the difference in elevation of the bases of adjacent footings, divided by the least horizontal clear distance between the footings, shall not exceed a value appropriate to the characteristics of the subsoil, and in general should not exceed 1.0. An increased load on the lower footing will result from this configuration.

3.7.4 Drainage

Unless underwater construction is specified, surface water or groundwater shall not be permitted to accumulate in excavations for footings. Such water shall be conducted to sumps located outside the boundaries of the footing and removed. If the water cannot be handled by this procedure, groundwater lowering should be accomplished by well points, a tremie seal coarse, or other appropriate means.

3.7.5 Treatment of Bottom of Excavation

Care shall be exercised to prevent disturbance of the materials at the bottom of the excavation by equipment or by the feet of the workmen. The bottom 2 in. of concrete in the footing shall be neglected for strength calculations.

On soft clayey or silty soils a working platform or mud coat of lean concrete, from 2 to 3 in. in thickness, is recommended if disturbance is probable. Otherwise, final excavation of the last 3 to 6 in. above grade should be deferred until immediately before placement of the reinforcement. The concrete in a working platform or mud coat shall not be considered as contributing to the strength of the footing.

If a tremie seal is to be placed to permit dewatering of the cofferdam for pier, the thickness of the seal shall be adequate to withstand the upward pressure of the water beneath the seal at the time of dewatering. This uplife force shall be determined by a rational analysis.

3.7.6 Stresses

Concrete and steel allowable stresses shall be in accordance with Reinforcement Concrete Design, Part 2 of this chapter.

3.7.7 Information on Drawings

Design drawings shall indicate the allowable soil pressure, type of soil, grade of the reinforcing steel, strength of concrete, minimum cement factor, and other pertinent data.

Part 10

Reinforced Concrete Culvert Pipe

Revised 1989

10.1 GENERAL

10.1.1 Scope

(a) These specifications cover the design and installation of reinforced concrete pipe for railway culverts.

10.1.2 Definitions

10.1.2.1 Pipe Installation Conditions

(a) Trench Installation — The pipe is installed in a relatively narrow trench excavated in undisturbed soil and then covered with backfill extending to the ground surface.

(b) Positive Projecting Embankment Installation — The pipe is installed on original ground or compacted fill with the top of the pipe above the ground, or compacted fill and then covered by embankment.

(c) Negative Projecting Embankment Installation — The pipe is installed within a relatively narrow trench with the top of the pipe below the natural ground, or compacted fill and then covered with embankment.

(d) Induced Trench Installation — The pipe is installed in a trench, backfilled with compressible material over the pipe, and then covered by a high embankment.

(e) Jacked or Tunneled Installation — The pipe is installed without removal of the ground above the pipe. Grouting of the exterior annular space around the pipe may be required to ensure full contact with the soil around the pipe. If existing soil conditions require an oversized tunnel, or if anticipated service conditions require access to the pipeline, a carrier pipe may be installed within the tunnel or casing pipe.

10.1.2.2 Symbols

B_c = Outside width of the pipe in the in-place condition (feet)

B_d = Width of the pipe trench (feet)

B_f = Bedding factor is defined as the ratio between the supporting strength of buried pipe to the strength of the pipe determined in the three-edge bearing test obtained according to the requirements of ASTM Designation C-497.

D = Inside span (or horizontal width) of the pipe (feet)

D-Load = The supporting strength of a pipe loaded under the three-edge bearing test conditions expressed in pounds per linear foot per foot of inside diameter or horizontal span.

FS = Factor of Safety (normally taken as 1.0)

H = Height of cover over the top of the pipe (feet)

p = *Projection Ratio* is defined as the vertical distance between the top of the pipe and the top of the trench divided by the trench width (negative projecting), or the height of the induced trench divided by the outside horizontal span of the pipe (induced trench).

w = Unit weight of the backfill material (lbs. per cubic foot)

W_L = Live Load including Impact transmitted to the pipe (lbs. per square foot)

W_E = Earth loads transmitted to the pipe (lbs. per linear foot)

W_S = Other loads transmitted to the pipe (lbs. per square foot)

10.2 MATERIALS

10.2.1 Pipe

(a) Pipe shall conform to the following ASTM Standards for type, size, shape, manufacturing, testing, and strength requirements as specified by the Engineer:

1. ASTM Designation C-76, Specification for Reinforced Concrete Culvert, Storm Drain and Sewer Pipe.
2. ASTM Designation C-506, Specification for Reinforced Concrete Arch Culvert, Storm Drain, and Sewer Pipe.
3. ASTM Designation C-507, Specification for Reinforced Concrete Elliptical Culvert, Storm Drain, and Sewer Pipe.
4. ASTM Designation C-655, Specification for Reinforced Concrete D-Load Culvert, Storm Drain, and Sewer Pipe.

10.2.2 Manholes

(a) Precast concrete manholes, if required, shall conform to ASTM Designation C-478, Specification for Precast Concrete Manhole Sections.

10.2.3 Rubber Gaskets

(a) Rubber gaskets, if required, shall conform to ASTM Designation C-443, Specification for Joints for Circular Concrete Sewer and Culvert Pipe.

10.2.4 Acid Resistant Coatings

(a) These coatings shall be specified by the Engineer for the particular condition required.

10.3 DESIGN

10.3.1 General

(a) The design of reinforced concrete pipe culverts must take into account the type of installation and bedding, the soil constants of the natural ground and backfill, the relative settlements of the pipe, pipe foundation, bedding, backfill and natural ground, acidity of the flow, the physical measurements such as depth of cover and width of cut, determination of earth load, live load, impact, and any additional loading.

10.3.2 References

Satisfactory design methods, utilizing more exact design procedures, are referenced for the use of the Engineer:

1. United States Department of Agriculture Soil Conservation Service Engineering Division Technical Release No. 5.
2. American Concrete Pipe Association Concrete Pipe Design Manual.
3. American Concrete Pipe Association Concrete Pipe Handbook.

10.3.3 Loads

(a) Design loading on the pipe shall include Earth Load, Cooper E 80 Live Load, Impact, and any other surcharge loads.

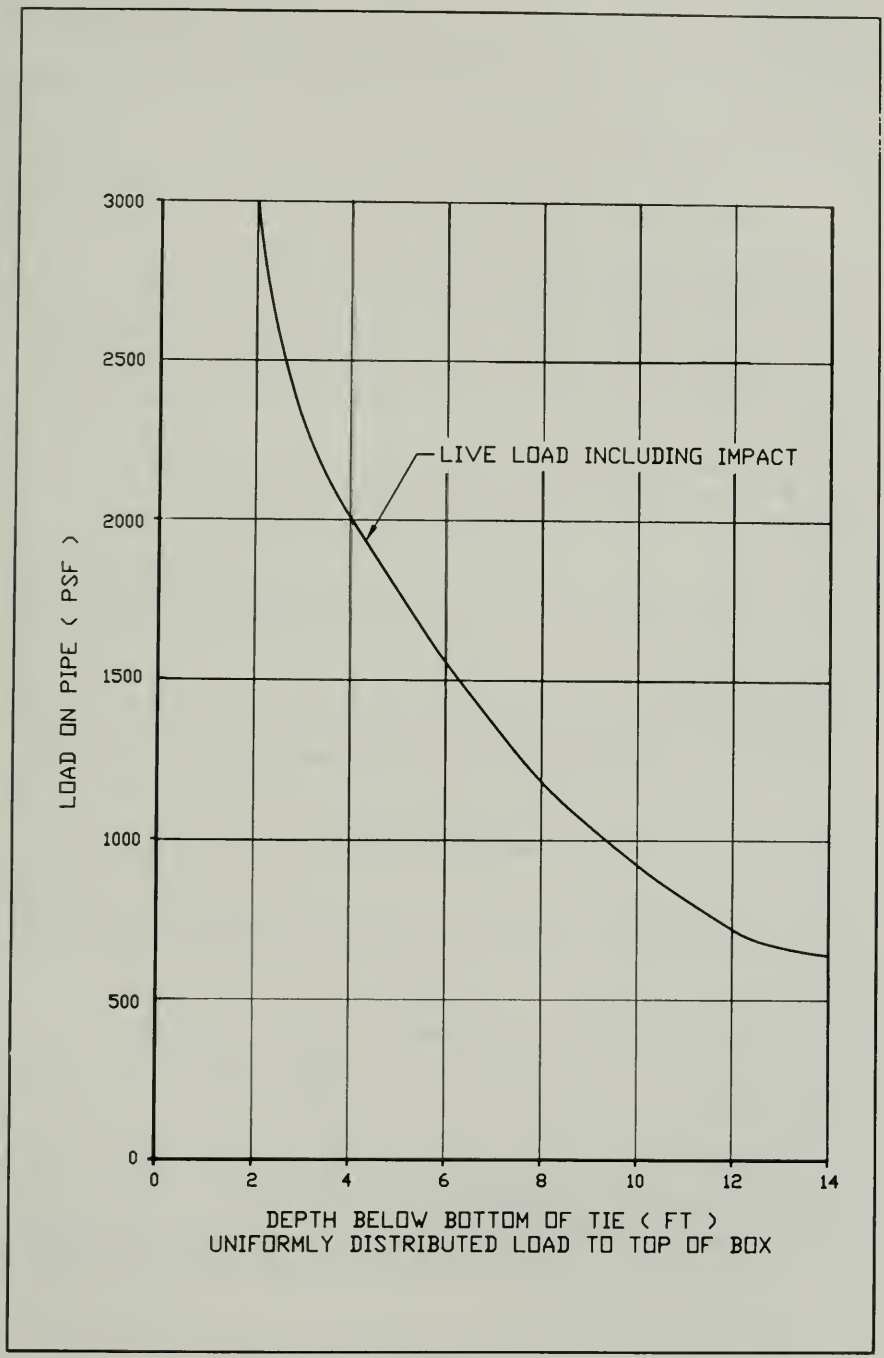

3000
2500
2000
1500
1000
500
0
LOAD ON PIPE (PSF)
LIVE LOAD INCLUDING IMPACT
0
2
4
6
8
10
12
14
DEPTH BELOW BOTTOM OF TIE (FT)
UNIFORMLY DISTRIBUTED LOAD TO TOP OF BOX

Figure 10.3.3

(b) Earth load tables from the references given in 10.3.2 may be used to calculate the weight of earth on the pipe. The earth load carried by the pipe is generally more than $B_c \times H \times w$.

(c) The Engineer may use the equations below in order to determine the earth load transmitted to the pipe. Other acceptable methods of analysis are given in the 10.3.2.

1. Trench Installation

$$W_E = 0.85 \times B_d \times H \times w$$

2. Positive Projecting Embankment Condition

$$W_E = 2.00 \times B_c \times H \times w$$

3. Negative Projecting Embankment Condition

$$W_E = 1.00 \times B_d \times H \times w$$

(d) Track Loading to be supported by the pipe is shown in Figure 10.3.3(c) of these specifications. The indicated loading includes a variable Impact Load of 40% at 0 feet, and 0% at 10 feet.

(e) Any surface surcharges, other than track load, shall be converted to additional height of fill to determine their loading on the pipe.

(f) Loading on a carrier pipe, that is within a casing pipe, shall be taken as the full Dead + Live + Impact Load without consideration of the presence of the casing, unless the casing pipe is permanently protected from corrosion using such means as providing extra pipe thickness or a resistant coating.

(g) The design trench width shall be indicated on the construction drawings as a maximum width of trench, or the Engineer shall design the pipe for the maximum effective trench width (transition width) as indicated in the Concrete Pipe Association Design Manual. The minimum width of the trench shall be $B_c + 2$ feet or $1.5\ B_c$ whichever is greater.

10.3.4 Bedding Factors

(a) Bedding factors to be used in the equation for determination of the the D-Load shall be obtained from Table 10.3.4 or as permitted within that table by more elaborate analysis.

TABLE 10.3.4

BEDDING FACTORS

Pipe Installation Methods	Bedding Class				**
	A	B	C	D	Tunnel
Trench	*2.8	1.9	1.5	1.0	—
Positive Projecting Embankment	*2.8	*2.0	*1.7	1.0	—
Negative Projecting Embankment	*2.8	*1.9	*1.5	1.0	—
Induced Trench	*2.8	*2.0	*1.7	1.0	—
Jacked or Tunneled					
Carrier Pipe	2.8	1.9	1.5	—	—
Casing Pipe	—	—	—	—	3.0
Any Pipe with less than 3′-0″ of cover between bottom of tie and the top of the pipe	1.5	1.5	1.5	—	—

*More elaborate analysis, using the procedures referenced in 10.3.2, can result in a more economical design with greater load factors.

**See Figures 10.4.2.2.A through E

10.3.5 Minimum Pipe Strength

(a) Pipe subjected to track loads shall have a minimum strength of D = 1350 (Class III) even if analysis indicates that a lower D-Load is satisfactory.

10.3.6 Factor of Safety

(a) The standard Factor of Safety of 1.0 against a 0.01 inch crack should be used for design unless the Engineer indicates that a higher Factor of Safety is required.

10.3.7 Alternate Design Procedure

(a) In lieu of carrying out the complete design analysis required by these specifications, the Designer may use Class V RCP for all sizes up to a height of cover of 14 feet; for greater heights of cover, the designer must make an analysis. For Elliptical or Arch Pipe where D = 3000 RCP is not available, a design analysis shall be made.

(b) The Engineer may specify the use of Class V RCP if he feels that the conditions of the site, or construction procedures, require this strength of pipe.

10.3.8 Pipe Strength

(a) The required D-Load of the pipe shall be determined by the following equation:

$$\text{D-Load} = \frac{(W_L \times B_C + W_E + W_S \times B_C) \times FS}{B_f \times D}$$

10.4 INSTALLATION

10.4.1 Preparation of Subgrade

10.4.1.1 Excavation

(a) Trenches shall be excavated in accordance with the bank stability requirements to a width sufficient to allow for proper jointing of the pipe and thorough compaction of the bedding and backfill material under and around the pipe. Where feasible, trench walls shall be vertical. A maximum trench width in conformance with the design assumptions, should be specified on the construction plans. Wide trenches generally require the use of stronger pipe, and require a more complete design analysis. The completed trench bottom shall be firm and cleaned for its full length and width.

(b) Where specifically requested the pipe trench bottom may be cambered longitudinally to provide for expected settlement. If camber of the pipe trench is required, the indicated camber must be shown on the plans.

(c) Where specified on the plans, the excavation for a pipe to be placed within embankment fill shall be made after the embankment has been completed to a specified height above the top of the pipe.

10.4.1.2 Foundation

(a) If the foundation is incapable of supporting the pipe loads, an adequate support shall be supplied by excavating the unstable soil and backfilling with compacted material, or by such other means as may be specified or approved by the Engineer.

(b) If the foundation is muck, or similar yielding material, the pipe shall be supported by piling, or by other such means as may be specified or approved by the Engineer.

(c) For Class B or Class C Beddings, the subgrade should be undercut and replaced with compacted granular material, if necessary, so that a firm foundation free of protruding rocks is provided. Special care may be necessary with Class A or other unyielding foundation to cushion the pipe from shock when blasting can be anticipated in the area.

10.4.2 Pipe Installation

10.4.2.1 Laying Pipe

(a) Pipe laying shall begin at the downstream end of the culvert. The bell or groove end of the pipe shall be placed upstream. No culvert shall be put into service until a suitable outlet is provided for the water.

(b) Elliptical pipe shall be placed with the vertical axis within 5 degrees of a vertical plane through the longitudinal axis of the culvert.

10.4.2.2 Bedding

(a) Pipe bedding and placement shall be specified to conform to one of the Bedding Classes illustrated in Figures 10.4.2.2A through 10.4.2.2E.

(b) When pipe cannot be placed on a prepared surface but must instead be placed on an unprepared surface, the bedding shall be considered to be Class D Bedding. Class D Bedding should only be used for emergency work, and is not permitted for permanent installations unless authorized by the Engineer. For typical Class D Bedding see Figures 10.4.2.2A and 10.4.2.2C.

10.4.2.3 Joining Pipe

(a) Pipe may be either bell and spigot, or tongue and groove design unless otherwise specified. When bell pipe is used, a shallow excavation shall be made underneath the bell of sufficient depth so that the bell does not rest on the bedding material.

(b) Pipe sections shall be joined so that the ends are fully entered and the inner surfaces are reasonably flush and even.

(c) Joints shall be made with either mortar, grout, rubber gaskets, plastic mastic compounds, or other combination of these types as approved and specified by the Engineer. Mortar joints in pipe that is jacked into place shall not be sealed with mortar until the culvert jacking is complete.

(d) In areas where a tendency exists for pipe sections to separate, suitable ties shall be fabricated and installed to prevent this separation.

(e) Endwalls or headwalls should be used for culverts under tracks and designed to resist pipe separation as well as to retain the embankment.

10.4.2.4 Water Tightness

(a) If water tightness is a problem, rubber gasketed pipe is recommended. When such joints are specified the pipe should be tested for infiltration or exfiltration as stipulated by the Engineer. The maximum rate of leakage shall conform to the following accepted requirements, or to other standards set forth by the Engineer:

Infiltration — 0.6 gallons per inch of diameter per 100 feet of pipe per hour.
Exfiltration — 0.6 gallons per inch of diameter per 100 feet of pipe per hour when subjected to an internal head of 2 feet, and increased by 10% for each additional 2 feet of head.

10.4.2.5 Culverts Carrying High Acid Fluids

(a) Where the PH of the conducted fluid is less than 4.5, the internal surfaces of the culvert should be protected from acid attack by a special permanent coating. The Engineer shall specify the type of coating and the means of application.

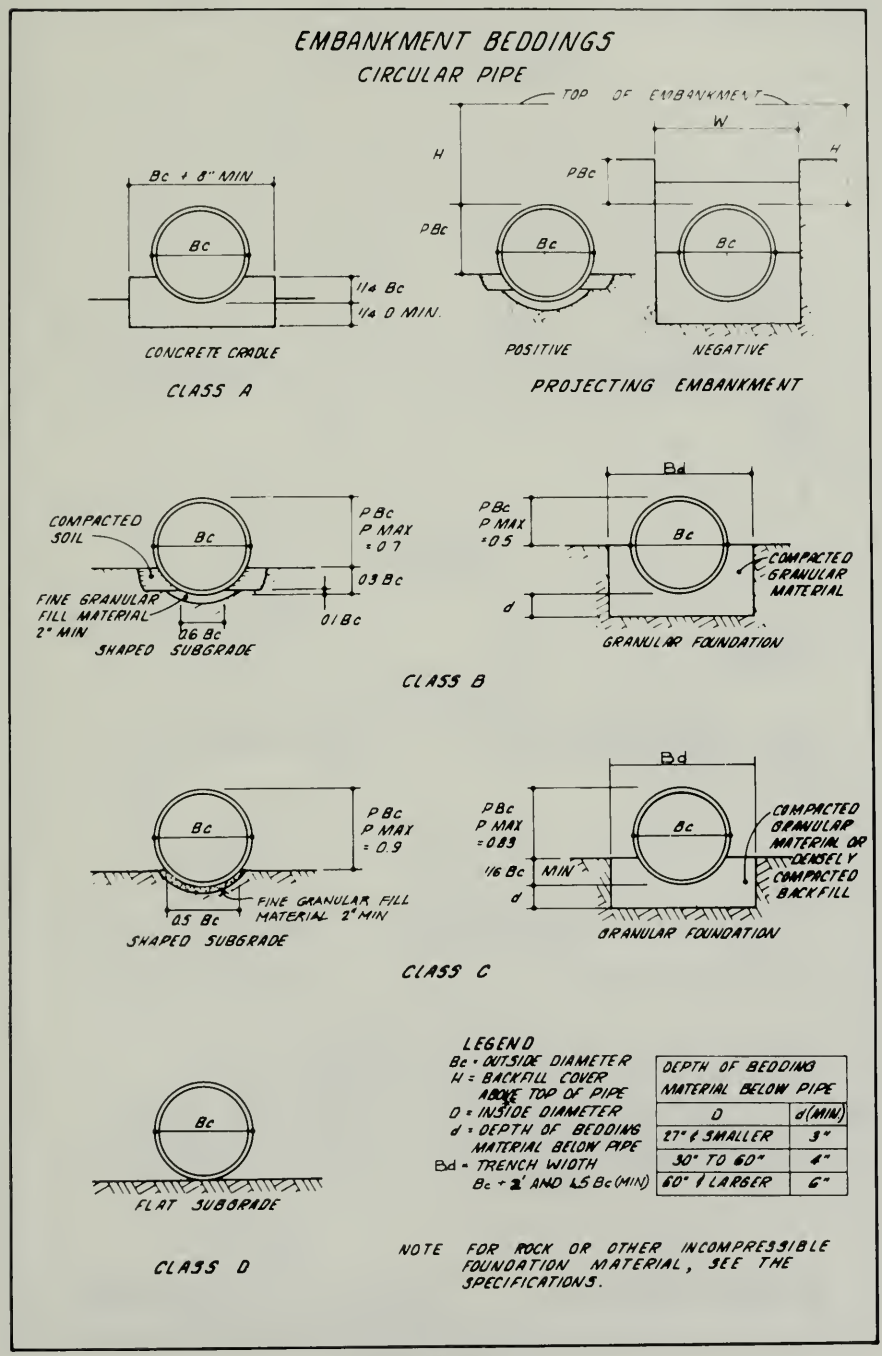


Figure 10.4.2.2A

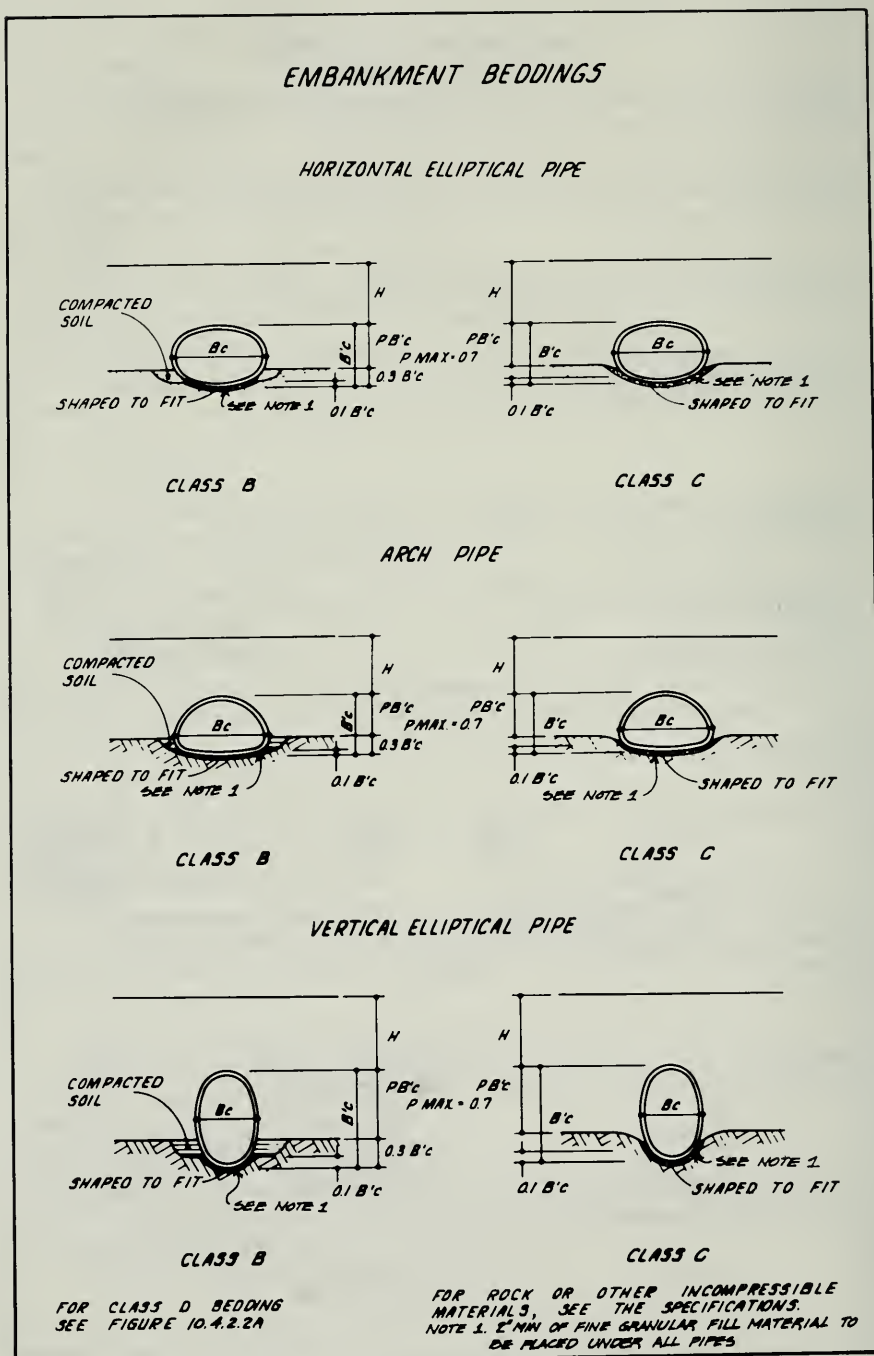


Figure 10.4.2.2B

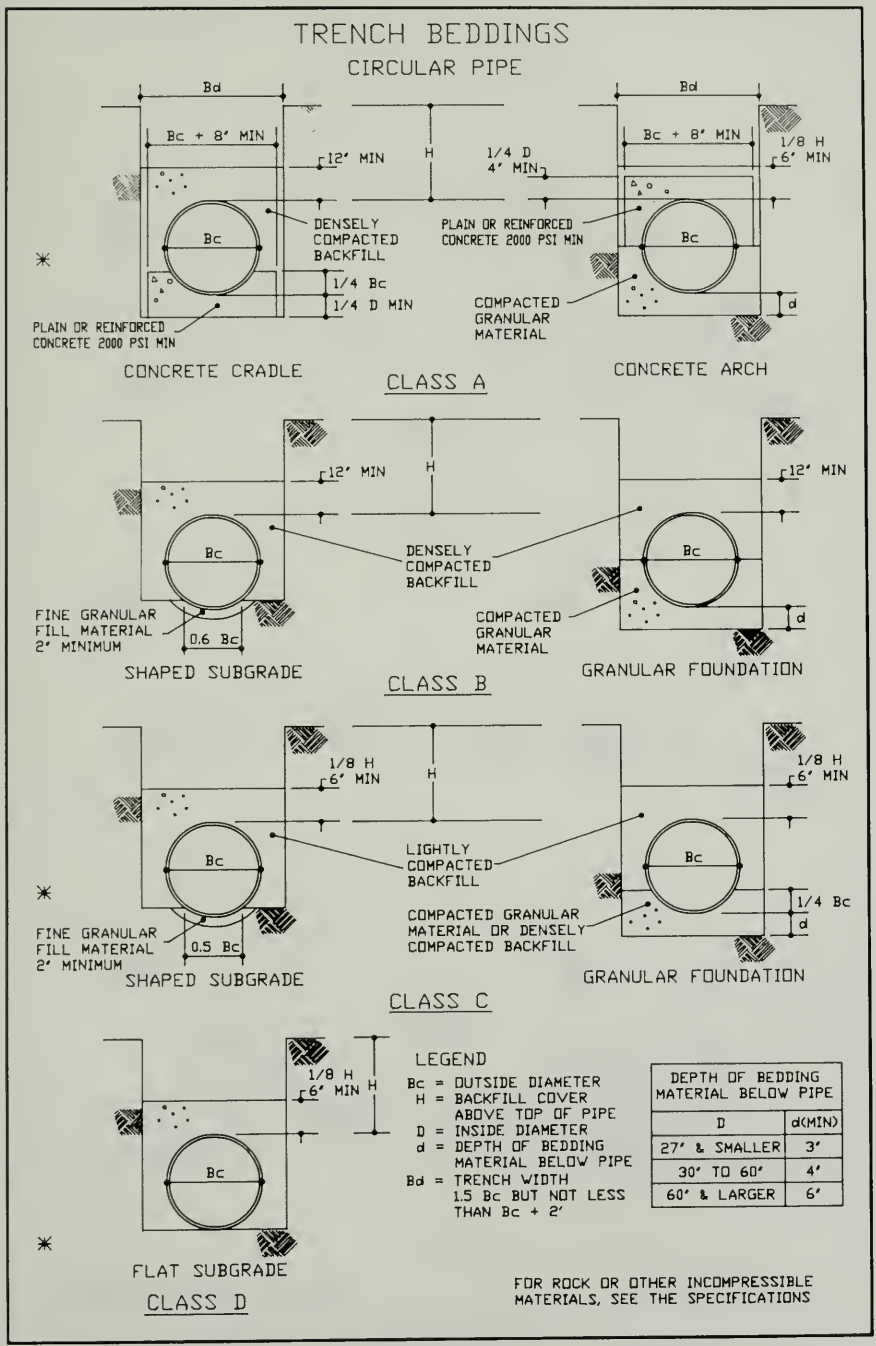


Figure 10.4.2.2C

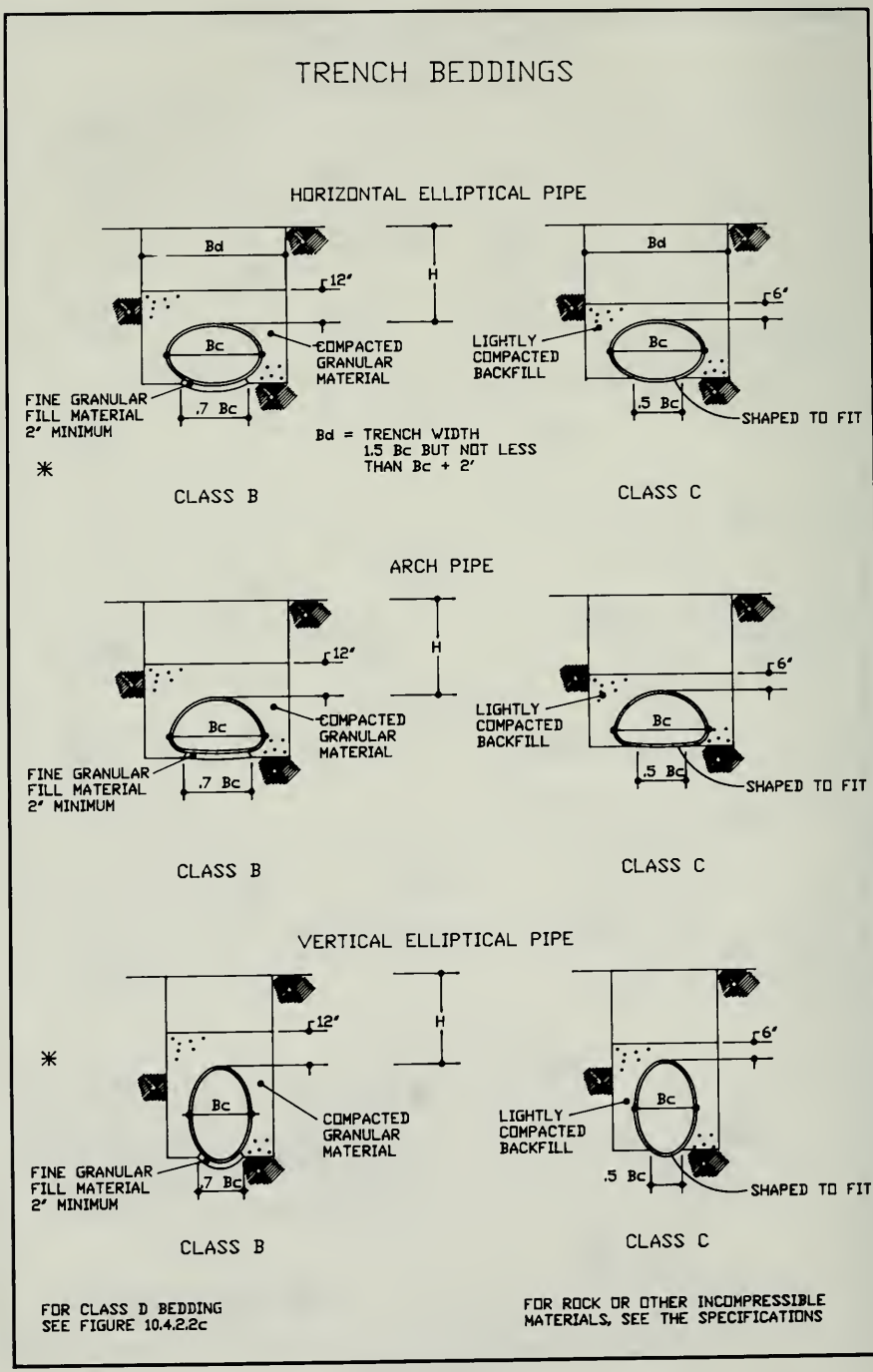


Figure 10.4.2.2D

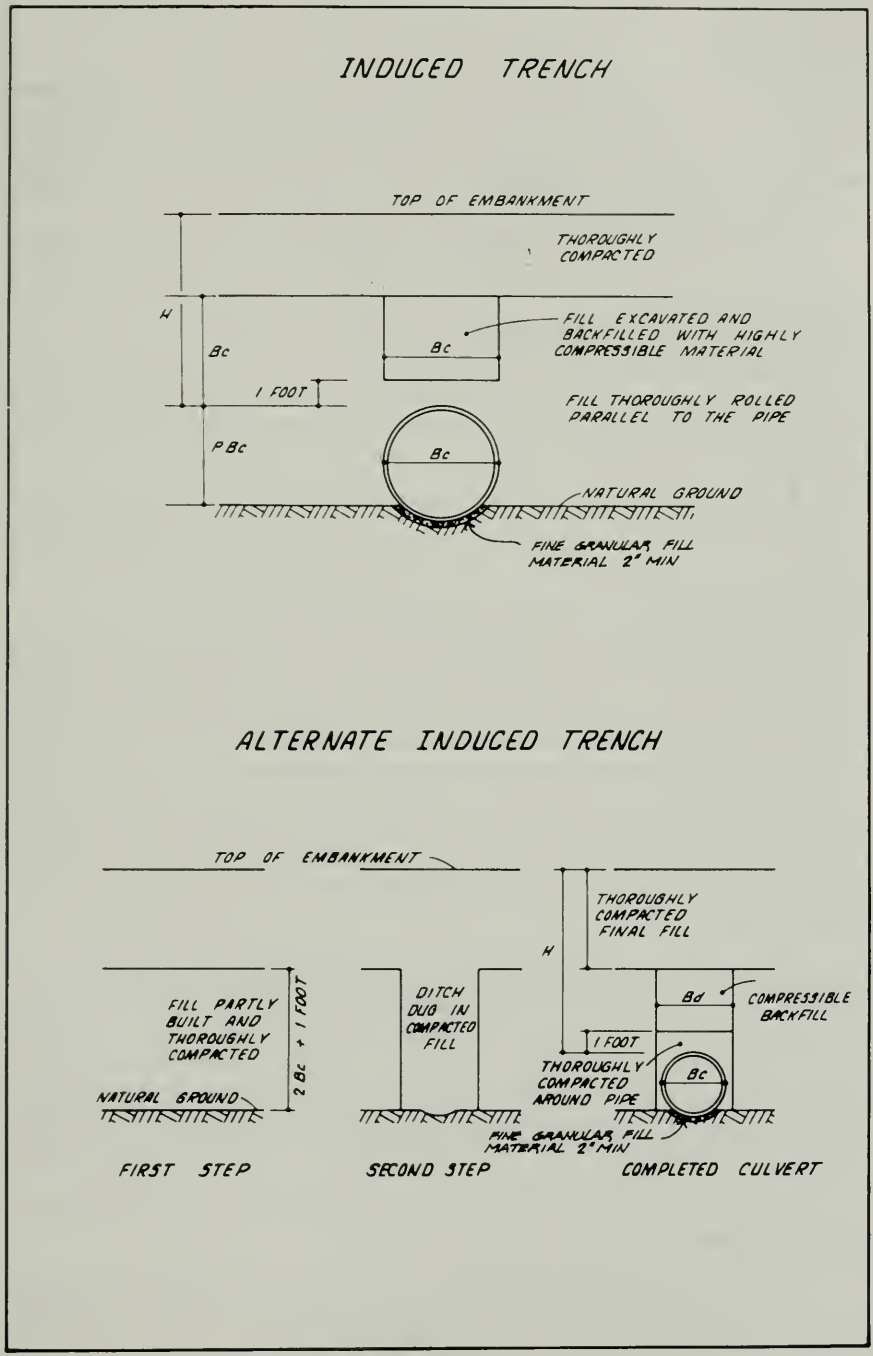


Figure 10.4.2.2E

10.4.3 Backfill and Embankment

10.4.3.1 General

(a) The backfill around the culvert shall be placed in accordance with the bedding requirements illustrated in Figures 10.4.2.2A through 10.4.2.2E, and other requirements of these specifications.

(b) All culverts that are to carry track load shall have the backfill thoroughly compacted to a minimum density of 95% as determined by ASTM D-698, and as specified elsewhere in the project specifications for adjacent embankment.

(c) Where the pipe is placed on a shaped subgrade, see Figures 10.4.2.2A, and 10.4.2.2C, extreme care shall be taken not to overexcavate the shaped surface so that point loading shall not occur on the pipe bottom.

10.4.3.2 Embankment Bedding (See Figures 10.4.2.2A and 10.4.2.2B).

(a) Where rock or noncompressible foundation material is encountered, the hard unyielding material should be excavated below the elevation of the concrete cradle (Class A) or the bottom of the pipe or pipe bell (Class B & C Beddings) for a depth of at least 6 inches or 1/2 inch for each foot of fill over the top of the pipe whichever is greater, but not more than 3/4 of the diameter (or horizontal span) of the pipe.

(b) For the Negative Projecting Embankment Condition, the width of the excavation, B_d, should be at least 1.5 x B_c and with a minimum of two feet greater than the outside diameter of the pipe for thorough filling and compaction of the void space under the pipe haunch.

10.4.3.3 Trench Bedding (See Figures 10.4.2.2C and 10.4.2.2D)

(a) Materials for backfill on each side of the pipe for the full width of the trench and to an elevation of 1 foot above the top of the pipe shall be fine, readily compacted soil or granular material, and shall not contain frozen lumps, stones that would be retained on a 2 inch sieve, chunks, highly plastic clay, or other objectionable material. Granular backfill material shall have 100% passing a 3/4 inch sieve, not less than 95% passing a 1/2 inch sieve, and not less than 95% retained on a No. 16 sieve. Oversized material shall be removed at the source of the material, except as directed by the Engineer.

(b) When the top of the pipe is even with or below the top of the trench, backfill material shall be placed at or near the optimum moisture content and compacted in layers not exceeding 6 inches (compacted) on both sides of the pipe for the full required length.

(c) Backfill material shall be placed and compacted for the full depth of the trench, unless induced trench installation is used.

(d) When the top of the pipe is above the top of the trench, backfill shall be placed at or near optimum moisture content and compacted in layers not exceeding 6 inches (compacted) and shall be brought up evenly on both sides of the pipe for its full length to an elevation 1 foot above the top of the pipe. The width of backfill on each side of the pipe for the portion above the top of the trench shall be equal to twice the diameter of the pipe or 12 feet whichever is less. The backfill material used in the trench section and the portion above the top of the trench for a distance on each side of the pipe equal to the horizontal diameter and to 1 foot above the top of the pipe shall conform to the requirements for backfill in 10.4.3.3 (a). The remainder of the backfill shall meet the requirements for embankment construction.

(e) The width of the trench, B_d, shall be a minimum of 1.5 x B_c and two feet greater than the outside diameter of the pipe in order to completely fill the void.

10.4.3.4 Induced Trench Bedding (See Figure 10.4.2.2E)

(a) The Induced Trench method shall not be used when the pipe is subjected to track loading without making a complete investigation of the settlements involved.

(b) When the Induced Trench method is used, the embankment shall be completed as required in 10.4.3.3 and as illustrated in Figure 10.4.2.2E, to a height above the pipe equal to the vertical outside diameter of the pipe plus one foot. A trench equal in width to the outside horizontal diameter of the pipe, in depth equal to the vertical outside diameter of the pipe, and to the length shown on the plans shall then be excavated to within one foot of the top of the pipe, trench walls being as nearly vertical as possible. This trench shall be loosely filled with highly compressible material. Construction of the embankment above the pipe shall then proceed in a normal manner using regular fill material.

(c) The length of the Induced Trench method shall be determined by the designer in keeping with the design assumptions and the pipe strength being used.

(d) When the Alternate Induced Trench method is used, the embankment shall be constructed in a normal manner to a height above the culvert bedding elevation equal to twice the outside diameter of the pipe. A trench as required shall then be excavated with the walls as nearly vertical as possible, and the pipe bedded and backfilled to one foot above the pipe as called for in 10.4.3.3. The remaining portion of the trench shall then be loosely filled with highly compressible material. Construction of embankment shall then proceed in a normal manner.

(e) In no case shall the length of compressible material extend to the ends of the culvert.

(f) Rock fill shall not be dumped over the culvert without a sufficient cushion of earth to prevent breakage of the pipe.

10.4.3.5 Jacking Pipe

(a) Pipe used for jacking through fills shall be tongue and groove design. The tongue shall preferably be at the downstream end. Jacking frames shall be so constructed as to avoid breaking the pipe or forcing it out of alignment. The pipe shall preferably be jacked upgrade in order to provide drainage at the heading during excavation. Satisfactory means shall be provided for maintaining the lead pipe at the correct line and grade.

(b) The pipe shall be installed according to specially prepared plans and specifications. The contractor shall set forth the construction procedure, extra pipe reinforcement and jack shield (if required), jacking pit location and shoring, and other special features for the safe and satisfactory completion of the work. Plans prepared by the contractor giving the construction details shall be submitted for review by the Engineer.

(c) Straw filler shall be inserted into voids created by excavation during jacking operations. Locations shall be recorded and after mining is completed, grout holes are to be drilled through the pipe and the voids filled with grout.

(d) A survey crew must continually monitor elevation and alignment of the railroad track above during the jacking procedures. Jacking must be stopped and any problems corrected if track movement is detected.

10.4.3.6 Constructing Pipe in Tunnels

(a) When it is necessary to place culvert pipe by tunneling, plans and specifications for the completed structure shall be prepared by the Engineer. The contractor shall set forth the construction procedures and other necessary details and submit them for review by the Engineer.

AMERICAN RAILWAY ENGINEERING ASSOCIATION

Part 19

Rules for Rating Existing Concrete Bridges

Revised 1989

19.1 GENERAL

19.1.1 Scope

(a) Except as otherwise provided in these rules, the Specifications for Reinforced Concrete Design or Prestressed Concrete Design, Parts 2 and 17, this Chapter, shall govern.

19.2 RATING

19.2.1 General

(a) Concrete railway bridges shall be rated according to their load carrying capacity as determined by the rules specified herein. The rating of the bridge shall be made either with reference to service loads and allowable service load stresses as provided in SERVICE LOAD RATING or, alternately, with reference to load factors and strength as provided in LOAD FACTOR RATING. The method to be used, SERVICE LOAD RATING or LOAD FACTOR RATING, shall be as directed by the Engineer. Once directed as to which procedure to follow, each bridge shall be assigned two ratings; NORMAL and MAXIMUM. Since different components of the bridge have different capacities, the rating of the bridge as a unit shall be the lowest of the ratings determined for the various components.

19.2.2 Normal Rating

(a) Normal rating is the load level which can be safely carried by the existing structure for an indefinite period of time without speed limitations.

19.2.3 Maximum Rating

(a) Maximum rating is the maximum load level which the structure can safely support at infrequent intervals.

19.2.4 Load Carrying Capacity

(a) The load carrying capacity of a bridge shall be determined by the computation of stresses or strengths based on actual records of the design, details, materials, workmanship, and physical condition, including data obtained by inspection (and tests if the records are not complete or it is deemed necessary). If deemed advisable, field determination of stresses or strengths shall be made and the results given due consideration in the final assessment of the load carrying capacity. For a specific service, the location, history and behavior of the bridge under investigation shall be taken into account.

19.2.5 Inspection

(a) Inspection of the bridge shall be made in accordance with Part 21, this Chapter, with special attention to the following items:

1. Whether the actual sections and details conform to the drawings. Actual dimensions of concrete sections should be measured to nearest 1/4 inch. Location and size of reinforcement should be checked at critical sections by use of a pachometer or other suitable device.
2. Any additions to the dead load not shown on the plans, such as heavier rail, deeper ballast section, walks, pipelines, conduits, signal devices, and wire supports.
3. The position of the track with respect to the center line of the bridge.
4. Any loss of concrete. All loose concrete should be removed before making this determination.

5. Any loss of steel due to corrosion, where steel has been exposed due to loss of concrete. This determination should be made by measurements after removal of corrosion.
6. The physical condition, noting such conditions as excessive cracking at points of maximum moment and at points of maximum shear. If cracks are deemed critical, they should be observed during the passage of a train to determine whether live load stresses cause movement or growth in the cracks.
7. Uneven settlement of supports.
8. Vertical or horizontal misalignment of spans or supports.

19.2.6 Computation of Stresses or Strengths

19.2.6.1 General

(a) The computation of stresses or strengths shall be made for the details, as well as for the main members, giving particular attention to the increased load carried by any beam, girder, slab or column due to the eccentricity of the load. This applies to bridges on tangent where the tracks are off center, as well as to bridges on curves.

19.2.6.2 Critical Sections

(a) The critical sections at which computations are made shall be determined by the Engineer. In addition to the main reinforcement, consideration shall be given to:

1. Sizes, spacing and anchorage of web reinforcement.
2. Anchorage of and splices in longitudinal reinforcement.
3. Column moments due to continuous construction or eccentricity of loading.
4. The bond (anchorage) requirements in effect at the time the bridge was designed.

19.2.6.3 Redistribution of Moments

(a) For continuous bridges, moments determined by elastic analysis may be redistributed by increasing or decreasing the negative moments by not more than 20 percent.

(b) This redistribution of moments should follow that given in "Building Code Requirements for Reinforced Concrete—ACI 318-83 ART. 8.4".

19.2.6.4 Maximum Reinforcement

(a) For LOAD FACTOR RATINGS the area of tension steel to be used in computing the design moment strength shall not exceed that available at the section, or 75 percent of the reinforcement required for balanced condition.

19.3 LOADS AND FORCES

19.3.1 General

(a) The loads and forces shall be computed and are defined as follows.

19.3.2 Dead Load

(a) The dead load shall be the weight of the bridge, including the track, ballast, and fill, together with that of any other fixed loads.

19.3.3 Live Load

(a) The live load shall be one of the Cooper E series, other standard loading, or a load consisting of a specific locomotive or other equipment, depending on the purpose for which the rating is desired.

(b) The lateral and longitudinal distribution of the axle loads to the structure shall be determined as specified in Specifications for Reinforced Concrete Design, Part 2, this Chapter.

19.3.4 Impact

(a) The impact shall be determined by the rules for impact in the Specifications for Reinforced Concrete Design, Part 2, or Prestressed Concrete Design, Part 17, this Chapter, whichever applies.

(b) Reduction of impact will be allowed as follows: For speeds less than 40 mph the impact shall be reduced in a straight-line variation from full effect at 40 mph to 0.5 of the full effect at 10 mph.

(c) Where impact is a function of live load and dead load, it may be necessary to recalculate the rating factors taking into account a revised impact percentage.

19.3.5 Other Loads

(a) Other loads shall be determined as given in Specifications for Reinforced Concrete Design, Part 2, this Chapter.

19.4 MATERIALS

19.4.1 Concrete

19.4.1.1 General

(a) The compressive strength of the concrete shall be taken as the 28-day strength of the concrete, if records of same are available. If there is no record of the compressive strength of the concrete, it shall be assumed as 3,000 psi for concrete which has not become badly weathered. If the concrete has become badly weathered, the compressive strength should be assumed as not more than 2,500 psi, and a lower value should be used if deemed necessary by the Engineer.

19.4.1.1.1 Coring for Strength Tests

(a) In the event that the concrete stress proves to be critical, the compressive strength of the concrete may be determined by "Standard Methods of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete", ASTM Designation C 42.

19.4.1.1.2 Treatment of Core Holes

(a) All core holes shall be filled with concrete having a 28-day specified compressive strength equal to or higher than that of the structure. Epoxy bonding compound, meeting the approval of the Engineer, shall be used to bond the new concrete to the existing concrete.

19.4.1.2 Service Load Rating

(a) The permissible stress for the concrete shall be taken as $1.2\,f_c$, where f_c is the allowable stress as specified in the Specifications for Reinforced Concrete Design, Part 2, this Chapter.

(b) Modular ratio n shall be determined as the ratio of elasticity of steel to that of concrete, E_S/E_C, as specified in the Specifications for Reinforced Concrete Design, Part 2, this Chapter.

19.4.1.3 Load Factor Rating

(a) The applicable concrete design assumptions shall follow those in Specifications for Reinforced Concrete Design, Section 2.31, this Chapter.

19.4.2 Reinforcing Steel

19.4.2.1 Service Load Rating

19.4.2.1.1 Known Yield Strengths

(a) The yield strengths of the reinforcement shall be as shown on the plans unless mill test reports of the reinforcement used in the structure are available. If the reports are available, the yield strength of the reinforcement used in determining the strength shall be the lowest yield strength shown for the size of bar used at the section being analyzed.

19.4.2.1.2 Unknown Yield Strengths

(a) If the yield strength is unknown the following shall be used where f_y is that value shown in 19.4.2.2.2.

1. Tension in flexural members: (with or without axial loads):
   - Structural-grade steel bars ..... $0.8\ f_y$
   - Intermediate-grade steel bars and Grade 40 bars ..... $0.7\ f_y$
   - Hard-grade steel bars and Grade 50 or Grade 60 bars ..... $0.6\ f_y$
   - Wrought iron ..... 20,000 psi
   - Wire mesh or cold-drawn steel wire not exceeding 1/2 inch in diameter when used in one-way solid slabs only ..... $0.7\ f_y$ but not to exceed ..... 34,000 psi
2. Tension in web reinforcement:
   - All grades of steel ..... 24,000 psi
   - Wrought iron ..... 20,000 psi
3. Compression in column verticals:
   - Structural-grade steel bars ..... $0.6\ f_y$
   - Intermediate-grade steel bars and Grade 40 bars ..... $0.6\ f_y$
   - Hard-grade steel bars and Grade 50 or Grade 60 bars ..... $0.5\ f_y$
   - Wrought iron ..... 20,000 psi
4. Compressive reinforcement in flexural members:
   - All grades of steel ..... 24,000 psi
   - Wrought iron ..... 20,000 psi
5. Compression in composite column:
   - All grades of steel ..... 24,000 psi
   - Wrought iron ..... 20,000 psi
6. Compression in combination columns:
   - Open-hearth or Bessemer steel ..... $[0.8\ f_y/18{,}000]\ [15{,}000\text{-}0.25\ (L^2/r^2)]$
   - Wrought iron ..... $[0.8\ f_y/15{,}000]\ [12{,}500\text{-}0.21\ (L^2/r^2)]$
7. Compression in pipe columns:
   - Open-hearth or Bessemer steel ..... $[18{,}000\text{-}70\ (L/r)]\ (0.8\ f_y^2\ /810{,}000{,}000)$

   Where L is the unsupported length of the column
   r is the radius of gyration of the steel section

19.4.2.2 Load Factor Rating

19.4.2.2.1 Known Yield Strengths

(a) The yield strengths of the reinforcement shall be as shown on the plans unless mill test reports of the reinforcement used in the structure are available. If the reports are available, the yield strength of the reinforcement used in determining the strength shall be the lowest yield strength shown for the size of bar used at the section being analyzed.

19.4.2.2.2 Unknown Yield Strengths

(a) If the yield strength is unknown the following shall be used:

Reinforcing Bars	Minimum Yield Strength f_y (psi)
Structural Grade or unknown before 1968	33,000
Intermediate Grade, Grade 40 or unknown after 1967	40,000
Hard Grade or Grade 50	50,000
Grade 60	60,000
Grade 75	75,000

Prestressing Steel	Minimum Yield Strength f_y (psi)
1. Strands (ASTM A 416)	
a) Grade 270	
1. Low-Relaxation	243,000
2. Stress-Relieved	230,000
b) Grade 250	
1. Low-Relaxation	225,000
2. Stress-Relieved	213,000
2. Wire (ASTM A 421)	
a) Low-Relaxation	212,000
b) Stress-Relieved	200,000
3. High-Strength Bar (ASTM A 722)	
a) Type I (Plain)	128,000
b) Type II (Deformed)	120,000

19.5 LOAD COMBINATIONS AND RATING FORMULAS

19.5.1 Loads and Forces

(a) The effects of loads and forces to be used in the evaluation formulas shall be expressed in consistent units (i.e. kips or foot-kips). The following symbols represent the effect, due to the load or force specified, to be considered in the rating of a concrete railroad bridge. The effects shall be determined as stresses for service load rating and as forces for load factor rating.

D = Dead Load
L = Live Load
I = Impact Load
CF = Centrifugal Force
E = Earth Pressure
B = Buoyancy
SF = Stream Flow Pressure

19.5.2 Notations

$SLRF_N$ = Service Load Normal Rating Factor
$SLRF_M$ = Service Load Maximum Rating Factor
$LFRF_N$ = Load Factor Normal Rating Factor
$LFRF_M$ = Load Factor Maximum Rating Factor
S_f = Permissible Stress—taking into account the allowance specified in Article 19.4.1.2.(a), this Chapter.
S_N = Nominal Strength taking into account assumptions as specified in Part 2, this Chapter.
ϕ = Strength Reduction Factor as specified in Part 2, this Chapter.

19.5.3 Formulas

19.5.3.1 Service Load Rating

19.5.3.1.1. Normal Rating ($SLRF_N$)

(a) The rating factor ($SLRF_N$) shall be computed in accordance with the following formula:

$$SLRF_N = \frac{[S_f/1.2] - [D + E + B + SF]}{[L + I + CF]} \qquad (19\text{-}1)$$

(b) The rating expressed in terms of Cooper E series shall be computed in accordance with the following expression. For example, if the live load used in (19-1) were for a Cooper E80 series, then the rating value would be:

$$\text{L. L. Normal Rating} = \text{SLRF}_N \times 80 \tag{19-2}$$

19.5.3.1.2 Maximum Rating ($SLRF_M$)

(a) This rating factor ($SLRF_M$) shall be computed in accordance with the following formula:

$$\text{SLRF}_M = \frac{S_f - [D + E + B + SF]}{[L + I + CF]} \tag{19-3}$$

(b) The rating expressed in terms of Cooper E Series for full speed shall be computed in accordance wtih the following expression. For example, if the live load used in (19-3) were for a Cooper E80 series, then the rating value would be:

$$\text{L. L. Maximum Rating} = \text{SLRF}_M \times 80 \tag{19-4}$$

This rating may be increased by reducing the speed over the structure. A reduction of impact as defined in 19.3.4 can then be used to recalculate the rating.

19.5.3.2 Load Factor Rating

19.5.3.2.1 Normal Rating ($LFRF_N$)

(a) The rating factor ($LFRF_N$) shall be computed in accordance with the following formula:

$$\text{LFRF}_N = \frac{\emptyset\, S_N - 1.1\,(D + E + B + SF)}{1.3\,[5/3\,(L + I) + CF]} \tag{19-5}$$

(b) The rating expressed in terms of Cooper E Series shall be computed in accordance with the following expression. For example, if the live load used in (19-5) were for a Cooper E80 series, then the rating value would be:

$$\text{L. L. Normal Rating} = \text{LFRF}_N \times 80 \tag{19-6}$$

19.5.3.2.2 Maximum Rating ($LFRF_M$)

(a) The rating factor ($LFRF_M$) shall be computed in accordance with the following formula:

$$\text{LFRF}_M = \frac{\emptyset\, S_N - 1.1\,(D + E + B + SF)}{1.3\,(L + I + CF)} \tag{19-7}$$

(b) The rating expressed in terms of Cooper E Series for full speed shall be computed in accordance with the following expression. For example, if the live load used in (19-7) were for a Cooper E80 series, then the rating value would be:

$$\text{L. L. Normal Rating} = \text{LFRF}_M \times 80 \tag{19-8}$$

This rating may be increased by reducing the speed over the structure. A reduction of impact as defined in 19.3.4 can then be used to recalculate the rating.

19.6 EXCESSIVE LOADING

19.6.1 Action to be Taken

(a) If the normal operating loads exceed those permissible under these rules, the speed and/or the loading shall be restricted so that the permissible loads will not be exceeded; otherwise, additional appropriate action should be taken until the bridge is strengthened or renewed. When the permissible loads are closely approached, or when the physical condition of the main members or the details are not good, the bridge shall be kept under frequent inspection as along as it is continued in service.

AMERICAN RAILWAY ENGINEERING ASSOCIATION

Part 13

Precast Concrete Box Culverts

Adopted 1989

13.0 SPECIFICATIONS FOR DESIGN, MANUFACTURE AND CONSTRUCTION OF PRECAST CONCRETE BOX CULVERTS

13.1 GENERAL

13.1.1 Scope

(a) These specifications govern the design, manufacture and construction of precast reinforced concrete box culverts supported on soil and rock foundations. Precast box culverts supported on pile foundations require special analysis.

(b) Cast-in-place concrete box culverts are covered in Part 16 of this Chapter.

(c) Except as noted otherwise, the provisions of Part 16, of this Chapter shall apply to this part.

13.1.2 Materials

(a) Minimum specified compression strength (f'_c) for precast reinforced concrete box culverts shall be 4,000 pounds per square inch at 28 days. Concrete shall be air-entrained when specified by the Engineer. The requirements for air-entraining admixtures and air content shall be as specified in Part 1, Sections 1.6 and 1.11, respectively, of this Chapter.

(b) Reinforcement steel shall conform to: ASTM Designation A 615, Grade 60; or welded steel wire fabric conforming to ASTM Designations A 185 or A 497; both with allowable tensile stress (f_s) of 24,000 psi for service load design.

(c) All hardware for sleeves, anchor bolts, inserts and other purposes shall be hot-dip galvanized in accordance with ASTM Designation A 153, stainless steel in accordance with ASTM Designation A 304, or epoxy coated in accordance with ASTM Designation A 775.

(d) Gasket material shall be rubber conforming to ASTM Designation C 443.

13.2 DESIGN PRINCIPLES

13.2.1 General

(a) The design principles specified in Part 16, Section 16.2, of this Chapter shall govern in addition to the following provision:

1. The design of the unit shall take into consideration stresses induced by handling during manufacture, transportation, and placement.

13.3 DETAILS OF DESIGN

13.3.1 General

(a) The details of design specified in Part 16, Section 16.3, of this Chapter shall govern in addition to the following provisions:

1. The cover of concrete over the circumferential reinforcement shall be 2 inches unless approved otherwise by the Engineer.
2. Wall thickness for the precast culvert shall be a minimum of 8 inches; top and bottom slab thickness shall be a minimum of 10 inches.

3. Where required, the culvert shall have a longitudinal camber of the magnitude specified on the plans.

13.3.2 Wingwalls

(a) Wingwalls may be cast-in-place or precast. If precast, it is recommended that the walls be monolithic with an apron.

13.3.3 Longitudinal Reinforcement

(a) The minimum longitudinal reinforcement in each face of the top slab, bottom slab and sidewalls shall be 0.2% of the cross-sectional area.

13.4 MANUFACTURE

13.4.1 General

(a) The precast reinforced concrete culvert shall be manufactured in steel forms and steam cured in accordance with Part 1 of this Chapter.

(b) Concrete shall be placed by the wet cast method when air-entrainment is specified. When air-entrainment is not specified the precast reinforced concrete culvert sections may be manufactured by the dry cast method if approved by the Engineer.

(c) Handling devices or holes shall be permitted in each box section for the purpose of handling and laying. Following installation all handling devices and holes shall be completely filled with a non-shrink grout, approved by the Engineer.

13.4.2 Manufacturing Tolerances

(a) Internal Dimensions — The internal dimension shall vary not more than 1% from the design dimensions. The haunch dimensions shall vary not more than 1/4 in. from the design dimensions.

(b) Slab and Wall Thickness — The slab and wall thickness shall not be less than 95% of that shown on the plans. A thickness more than that required in the design shall not be cause for rejection.

(c) Length of Opposite Surfaces — Variations in laying lengths of two opposite surfaces of the box unit shall not be more than 1/8 in./ft. of span with a maximum of 5/8 in. in any box unit, except where beveled ends for laying on curves are specified by the Engineer.

(d) Length of Precast Unit — The underrun in length of a precast unit shall not be more than 1/8 in./ft. of length with a maximum 1/2 in. in any box unit.

(e) Position of Reinforcement — The maximum variation in the position of the reinforcement shall be $\pm$ 1/2 in. In no case, however, shall the cover over the reinforcement be less than 1-1/2 in., as measured to the internal surface or the external surface of the completed box unit unless approved otherwise by the Engineer. The preceding minimum cover limitation does not apply at the mating surfaces of the joint.

(f) Area of Reinforcement — The areas of steel reinforcement shall be the design steel areas as shown on the plans. Steel areas greater than those required shall not be cause for rejection. The permissible variation in diameter of any reinforcement shall conform to the tolerances prescribed in the ASTM specification for that type of reinforcement.

13.4.3 Physical Requirements

(a) When concrete is placed by the wet cast method concrete compressive strength shall be determined from cylindrical concrete specimens made in conformance with ASTM Designation C 39, and prepared in conformance with ASTM Designation C 31.

(b) When units are manufactured by the dry cast method cylinders shall be made in conformance with ASTM Designation C 361, Section 10.3.2.

(c) At least five test cylinders shall be prepared from each day's production of concrete.

(d) Compression test requirements shall be in accordance with ASTM Designation C 361, Article 10.3.3.

13.4.4 Marking

(a) The following information shall be clearly marked on each box unit by indentation, waterproof paint, or other approved means:

1. Date of manufacture;
2. Name or trademark of the manufacturer;
3. Identification of the plant; project name; location number; and top slab.

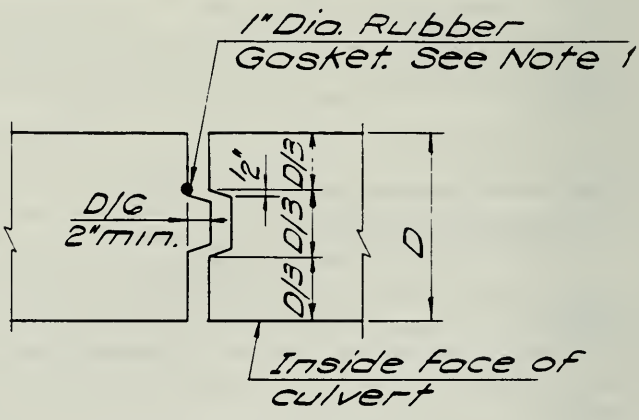


Fig. 13.5.1A

NOTE 1:
Rubber Gasket to be set in recessed groove with adhesive.

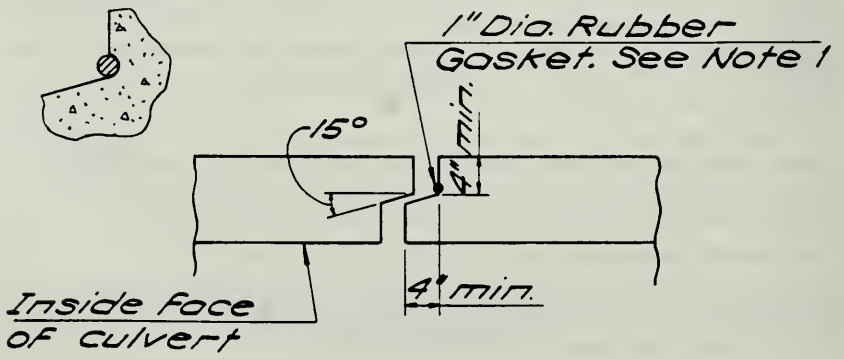


Fig. 13.5.1B

13.5 CONSTRUCTION

13.5.1 Joints

(a) The units shall be produced with tongue and groove or male and female ends such as shown in Figs. 13.5.1A and 13.5.1B or as approved by the Engineer. The ends of the units shall be of such design and so formed that when the units are laid together they will make a continuous line of box units with a smooth interior free of appreciable irregularities in the flow line, all compatible with the permissible variations given in Article 13.4.2.

(b) The joint shall be of an approved design and, where differential deflection from live load between units exceeds L/800, (where L is the width of the box) shall be capable of transferring shear loads through the top slab of adjacent units by a method or devices mutually agreed upon by a box culvert manufacturer and the Engineer. If individual shear connectors are used to fasten the adjacent top slabs together, they shall be spaced no more than 30 in. on center and with a minimum of two shear connectors per joint.

(c) The precast unit shall be placed against a previously completed unit in such manner as to assure an adequate seal.

(d) When the precast reinforced concrete units are used in parallel for multicell installations, positive means of lateral bearing shall be provided by grouting with non-shrink grout between the units or by filling the space between adjacent units with compacted granular material.

13.5.2 Foundations

(a) A compacted crushed stone bed shall be provided under the precast reinforced concrete box culvert. The depth of the crushed stone bed shall be a minimum of one foot, and shall extend one foot on each side of the precast reinforced concrete box culvert with a minimum one to one side slope as shown on Fig. 13.5.2.

COMMENTARY

13.4.1 (b) Air-entrainment is normally desired when units will be subjected to freeze-thaw cycles. Since the dry cast method is not compatible with air-entrainment the Engineer should consider this when preparing the specifications.

13.4.3 (a&b) The preparation of cylinders for determining concrete compressive strength differs for wet cast and dry cast concrete. The Engineer should determine the methods employed by potential manufacturers when preparing the specifications.

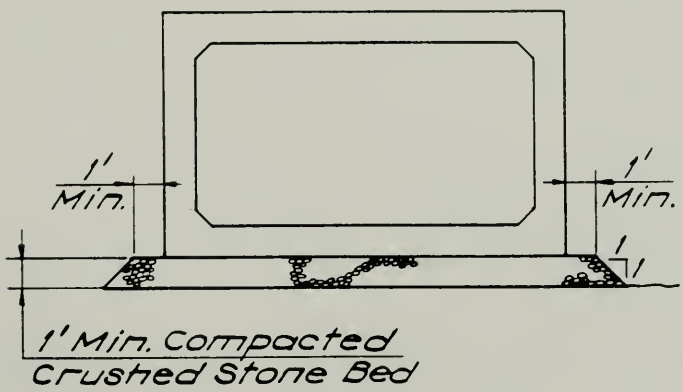


Fig. 13.5.2

Proposed 1989 Manual Chapter
Chapter 12 — Rail Transit

The following Foreword is proposed as Committee 12's initial entry to Chapter 12. Additional information for this chapter is being developed and will be submitted in future revisions.

Foreword

Chapter 12 — Rail Transit is intended to serve as a guide and offer insight for planning, design and maintenance of a rail transit system. Recognizing the importance of the technical considerations required in the design, construction and maintenance of a rail transit system, the well informed choices of the planner are of utmost importance. The available methods, components and applications are many and varied, therefore concise and clearly defined criteria are essential when developing goals and objectives. This chapter endorses the A.R.E.A. recommended practices that apply, and recognizes that many of the topics within require extensive coordination with other chapters in the Manual. Various techniques, components and methodologies for construction and maintenance must be evaluated fairly and accurately in order to consider alternative concepts on an equal basis. The reader is encouraged to evaluate the methods and effective practices utilized by the many operating rail transit systems in North America, and to utilize this Manual and chapter as a basis for the work.

Proposed 1989 Manual Revisions To Chapter 14 — Yards and Terminals

A new Section 4.3 on "Automobile and Truck Loading/Unloading Facilities" is proposed to replace the current section on "Automobile and Truck Transport."

4.3 AUTOMOBILE AND TRUCK LOADING/UNLOADING FACILITIES

4.3.1 Automobile Loading/Unloading

4.3.1.1 General

The transfer of automobiles to and/or from rail cars (i.e., rail-truck, rail-rail, truck-rail, ground-rail or rail-ground) generally requires a separate facility to accomplish this task. This separation is required to insure protection and security of the automobiles.

Domestically-produced automobiles are frequently loaded on rail cars within the confines of the manufacturing plant. Foreign-produced automobiles are usually loaded on rail cars at dock-side. Many transfer variations can be utilized; however, in each case, securing the operation is of prime concern. This type of facility may also be used for automobile ferry services.

Items to be considered in selecting an automobile loading/unloading (transfer) facility are:

1. **LOCATION**—A site should be selected with easy access to main highways, as well as ease of rail switching. Consideration should also be given to the potential of vandalism so as to avoid missile damage and theft. Proximity to areas that may generate air pollution which could damage automobile finishes should also be examined. It is also desirable to not locate in low lying areas to avoid potential flood damage to vehicles.

2. **SIZE**—The size of the facility, its trackage, ramping and vehicle storage areas, should be large enough to handle the maximum expected load under the proposed operating conditions. Some of the conditions to be considered are: the average work week, type and quantity of vehicles handled and the number of agencies using the same facilities. The auto production and distribution process, by its very nature, requires a considerable degree of advance planning, including volume predictions. All auto manufacturers can and do make rather good volume predictions which can be utilized for planning purposes. Future needs should also be evaluated if possible.

Factors which affect the sizing of an unloading facility are:

a) Automobile Parking—Requires estimate of maximum number of vehicles in facility at one time.
b) Truck Transport Area—The number of truck transport loading stalls must be determined.
c) Track Capacity—Adequate multi-level rail car capacity should be provided to allow one switch or spot per shift.
d) Rain Runoff Retention Ponds—Consideration should be given to retaining runoff from the large paved areas to prevent flood damage to adjoining properties.
e) Buildings—Office space, washrooms and locker facilities should be determined. Cleanliness of employees handling automobiles is important to prevent soiling of vehicles. Office buildings or servicing facilities for haul-away trucks may be necessary. However, separation of these facilities from the primary automobile area is desirable to maintain automobile security.
f) Employee/Visitor Parking—Consideration should be given to providing a separate secured parking area.
g) Miscellaneous—Dead battery parking, damaged vehicle parking, specialized vehicle handling requirements (e.g., van, truck, luxury and military).

3. **SECURITY**—Since the vehicles are left with the keys in them, security is of prime importance. The entire area should be fenced to discourage unauthorized entry and theft. Many facilities are arranged so that automobiles can only be driven out of the parking area over haul-away docks or ramps. Provisions for checking employees and visitors in and out should be made. Parking stalls should be away from security fence to provide an unobstructed buffer area around inside of fence. Other electronic security devices can also be utilized.

4. **LIGHTING**—Lights should be provided for entire area adequate for security and for loading, unloading and inspection, if required. Poles should not be located in fence line.

5. **ZONING, BUILDING CODES AND PERMIT REQUIREMENTS**—All Local, State and Federal requirements must be met. Considerable time may be required to obtain some Permits.

6. **SPECIAL REQUIREMENTS**—Customers' needs or standards should be evaluated.

4.3.1.2 Design Considerations

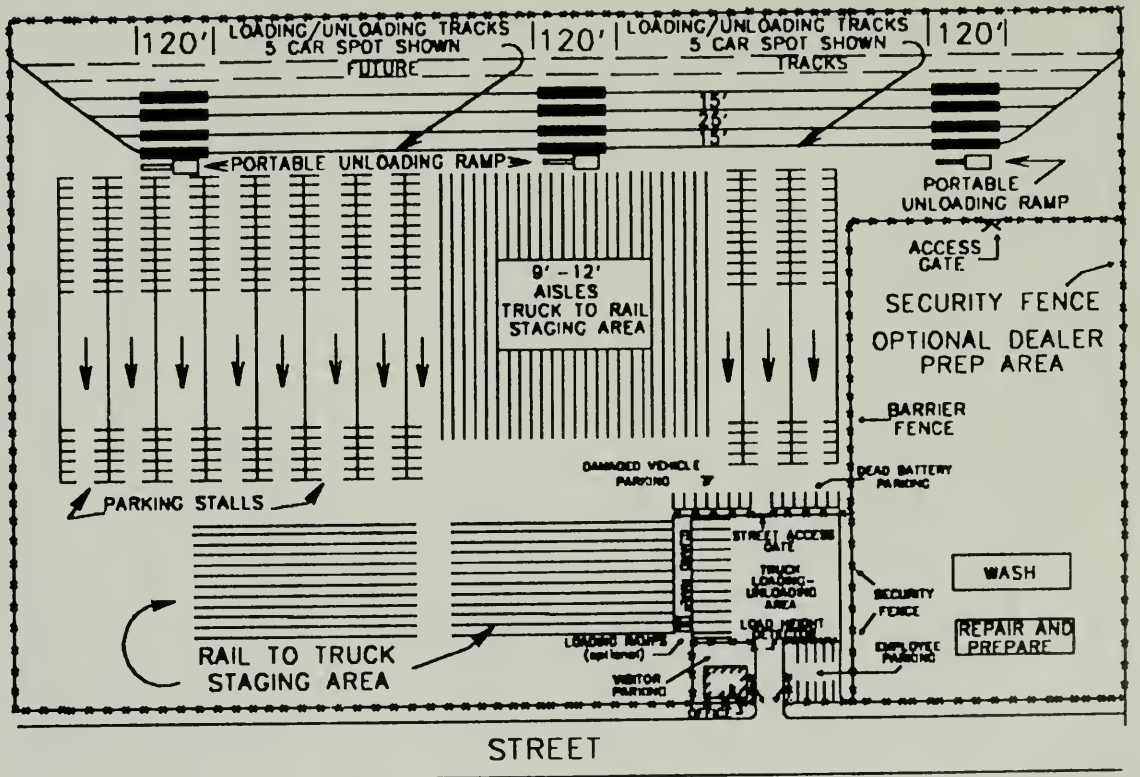
1. **PARKING**—Layout configurations vary from facility to facility. Most patterns are determined by the overall size and shape of the land available. Four patterns of parking are available. They are 90 degree-head to head, angle parking-head to head or angle (herring-bone) pattern. Parallel line parking, head to tail, is sometimes utilized for direct loading. The advantages or disadvantages of each must be addressed for each facility.

a) Typical Configurations (right angle parking-head to head)
(See Site Plan Example—Figure 7)
(1) Stalls —
Standard — 10 ft. x 20 ft.
Luxury or Van — 10 ft. x 22 ft.
Dead Battery — 11 ft. x 20 ft.
Damaged Vehicles — 12 ft. x 22 ft.
Clearance from security fence — minumum 10 ft.
(2) Aisles —
Between Stalls, One Way — minumum 22 ft.
Main Trafficways, Two Way — minimum 30 ft.
(3) Number of stalls —
Approximately 125 vehicles per acre with 10 ft. x 20 ft. stalls.

b) Surface
Asphalt Paving Recommended —
Many facilities are built with 4 inch asphalt thickness made up of 2-1/2 inch base course and 1-1/2 inch surface course on a suitable soil or rock sub-base.
However, pavement design, including sub-base, should be based on local design considerations with asphalt thickness consistent with site conditions.

b) Grade —
As near level as practicable but with sufficient slope to promote drainage.
Sub-Grade —
Consistent with local conditions and adequate to support automobile loading.

c) Row and Stall Identification
(1) Striping recommended for parking stalls and lanes. Direction arrows may also be desirable.
(2) Rows designated with Capital Letters —
Desirable to locate row letters on signs (12 inch high letters) at end of rows, minimum 8-ft. to bottom of sign.
(3) Stalls numbered with numbers placed on left near aisle.



AUTOMOBILE LOADING/UNLOADING SITE EXAMPLE
Figure 7

d) Staging Area — Bumper to bumper staging (aisles 9 ft.-12 ft. wide) may be used for common destination movements.

2. **TRACK**

a) Grade —
As near level as practicable.

b) Centers —
(1) 15 ft. minimum
(2) 25 ft. or more where service vehicles drive between rail cars and for placement of bridge plate storage racks.

c) Structure —
(1) Open (unpaved)
(a) Advantages
(1) Ease of maintenance
(2) Easier drainage if ballast above pavement
(b) Disadvantages
(1) Requires crossings and/or unloading areas every four to six car lengths and unloading configuration is fixed
(2) Ballast must be kept out of roadways

(2) Paved
(a) Advantages
(1) Can spot rail cars for unloading from either end in any grouping (from one to six cars)
(2) Less restricted driving when tracks unoccupied
(b) Disadvantages
(1) Track maintenance difficult
(2) Requires sub-drainage
(3) Additional cost

d) Lengths —
(1) All (un)loading tracks same capacity if possible for uniformity of switching (approximately 95 track feet required for typical rail car)
(2) Multiple of four to six car segments with paved (or planked) unloading area (120 ft. minimum length) at each end and between each four to six car segments.
(3) Tangent track required for each multiple rail car spot.

e) Turnouts —
(1) Unpaved for ease of maintenance.
(2) Recessed switch stands where high switch stands might be struck by vehicles.

f) Limits for Rail Car Spotting —
(1) Stripe in paved areas.
(2) Steel bumping posts with heavy impact break-away design.
(3) Wheel stops placed for cars with least end overhang.
(4) Concrete dock with wood or mechanical bumper at coupler level.

g) Other Features
(1) Auxiliary trackage
(a) Secondary rail car storage tracks may be necessary, depending on volume of automobile shipments.
(b) Additional tracks may also be needed for rail car inspections and/or repairs prior to loading.

(2) Bridge plate storage racks
  (a) Serve two adjacent tracks
  (b) Should be opposite car couplers
  (c) Can also serve as mounts for fire extinguishers
  (d) Paint bright color to inhibit vehicles striking.

(3) Protection of (un)loading personnel
  (a) Provide Blue Flag protection at entry ends of all (un)loading tracks with rack for unused flag storage.
  (b) Provide private locks on entry switches, derails and/or gates.
  (c) Provide audible rotating light alarms on rail entry gates.

3. **RAIL CAR LOADING/UNLOADING EQUIPMENT**—Should be designed to allow quick drive on/off of automobiles. Automobile manufacturers should be consulted to determine maximum permissible ramp angles.

a) Typically self-propelled, rubber-tired, ramps used which can be raised and lowered to reach the three levels of a standard rail car.

Consideration should be given to provide concrete pavement or planked crossing surface at loading/unloading area under portable ramp since area is heavily travelled, has frequent twisting of wheels and is exposed to hydraulic oil and gasoline leaks.

b) Track or rail-mounted ramps used at same locations either from ground level or on elevated dock.

4. **TRANSPORT TRUCK LOADING/UNLOADING AREA**—The truck haul of automobiles is normally handled by a separate company or contractor. There are many variations of transport trucks in existence. It is important to verify transport truck dimensions before designing this area.

a) Provide sufficient space to permit turning and spotting of truck for loading/unloading of automobiles.

b) Volume predictions required to determine number of transport truck spaces. Stalls normally 12 ft. wide.

c) Consider use of concrete pavement for durability.

d) Provide physical separation of area from automobile parking-staging area for security. Fence and/or low guard rail normally used. Guard rail must be low enough to allow placement of ramps over rail and high enough to prevent driving automobiles over the rail. Verify transport truck dimensions.

e) Ground mounted, adjustable, steel ramps sometimes required to load some types of transport trucks.

5. **SECURITY—**

a) Fencing — Normally 8-ft. high industrial type chain link fence with top barbed wire or razor ribbon and bottom tied. All gates to be heavy duty, equipped with drop bars, welded hinges and adequate locking devices.

b) Crash or Highway Type Guard Rail — Recommended on inside of fence around perimeter of parking area to prevent vehicles from being driven through fence.

c) Electronic Surveillance — Television cameras, motion detectors or other electronic security devices should be evaluated.

d) Segregation of areas of responsibility with barrier fence.

6. **LIGHTING —**

a) Low intensity for security (minimum 1 foot candle with 1.5 foot candle average)

b) Higher intensity for loading/unloading operations (approximately 4 to 5 foot candles)

c) Locate all light poles away from security fence.

4.3.2 Truck Chassis Loading/Unloading

4.3.2.1 General

Factors regarding location, size, buildings, surfacing, security and lighting enumerated above for automobiles apply equally as well to truck chassis. The rail equipment and the placement of the trucks on the rail equipment differs. Trucks with cabs, but without bodies (truck chassis) are commonly shipped in "saddleback" fashion on a specially equipped flat car. Thus, the use of a crane is required for loading and unloading. While the loading may be done at a plant site exclusively devoted to trucks, the unloading operation can conveniently be incorporated into and made a part of a typical automobile unloading facility.

4.3.2.2 Unloading Track

Truck shipping volumes being considerably less than autos, a single track set apart from, but adjacent to, auto facilities should suffice. Volume and economic considerations will dictate the degree of separation from, and/or incorporation within, auto facilities.

4.3.2.3 Unloading Facilities

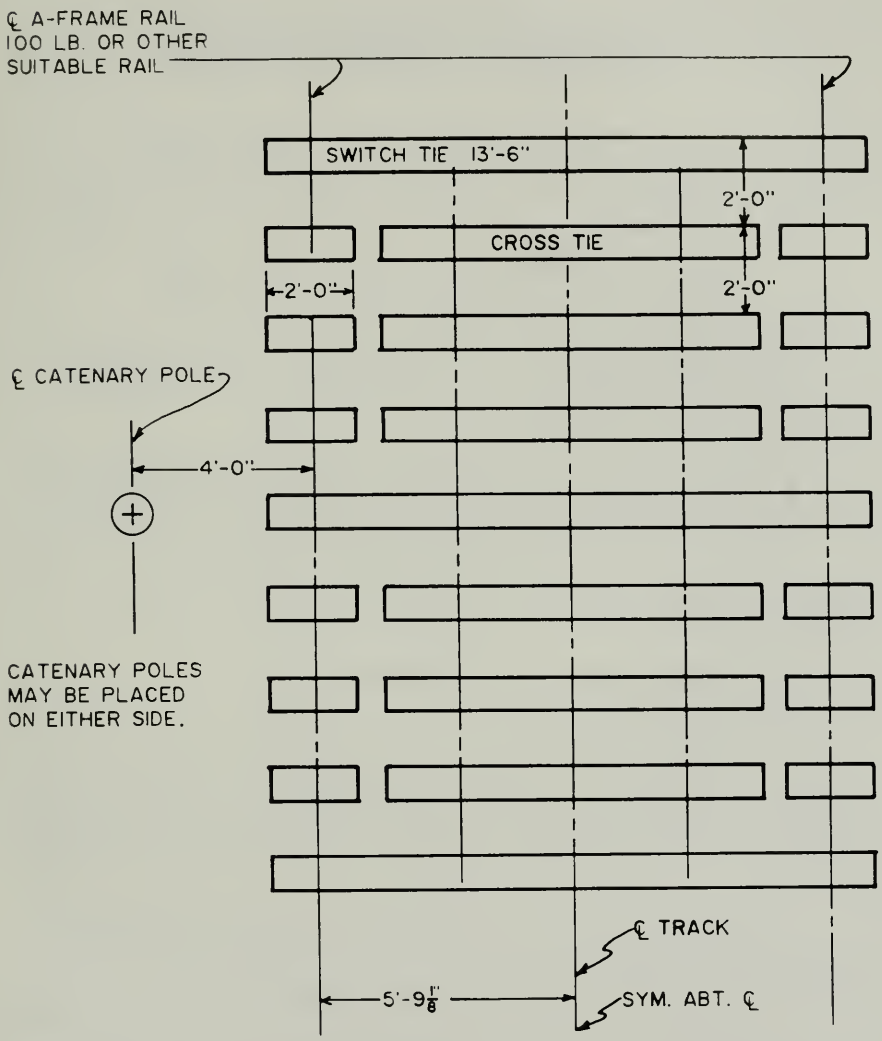
Trucks loaded in "saddleback" fashion must be removed from the truck they have been set upon and secured to for transport to a level position on the car deck before being started and driven from the car. The job can be accomplished by a mobile crane of sufficient capacity operated adjacent to the rail car where volume is light and the need only occasional. Where volumes require a greater degree of specialization, it is recommended that an "A" frame crane, track-mounted and electrically operated with running rails located outside of regular track rails, be provided. The "A" frame straddles the car to be unloaded and can be positioned to handle any car spotted within its reach. Fig. 8 details a tie layout to accommodate the "A" frame. Access to the unloading track for pre-starting service should be given consideration. Air supply sufficient to release truck brakes is a necessity.

4.3.3 Military Vehicles

This type of facility may also be used for receipt and transfer of new military vehicles. Special provisions may be necessary to handle some of these vehicles due to weight, size or other considerations.

4.3.4 Containerized Shipping

Some new automobile shipments are also being moved in standardized containers or enclosed trailers. These shipments can be moved directly from loading point to the dealer's site without actually handling the automobile while in transit. A TOFC-COFC facility is more appropriate for these types of shipments rather than the above-noted configurations.



NOTE: A-FRAME CRANE ELECTRICALLY PROPELLED. HAS DOUBLE FLANGED WHEELS. TOTAL WIDTH: 15'-0". SPACE REQUIRED FOR THIS FACILITY: 17'-3".

Fig. 8 — Typical layout for "A" frame truck unloading.

Proposed 1989 Manual Revisions To Chapter 15 — Steel Structures

The following changes are proposed to various Articles and Tables in Chapter 15:

Article 1.4.1 (pages 15-1-20 and 15-1-21) and **Article 2.4.1** (pages 15-2-7 and 15-2-8):

Shear in A 325 bolts	17,000*
Shear in A 490 bolts	21,000*
Bearing on A 325 and A 490 bolts	$\frac{L F_u}{2d}$ or 1.2 F_u**

Change the last sentence of the footnote marked "*" to:

"Where the Engineer has specified special treatment of surfaces or other than standard holes in a slip-critical connection, the allowable stresses in Table 9.1.4 of Part 9 may be used if approved by the Engineer."

Add a footnote marked "**" as follows:

**For single bolt in line of force or connected materials with long slotted holes, 1.0 F_u is the limit. A value of allowable bearing pressure on the connected material at a bolt greater than permitted can be justified provided deformation around the bolt hole is not a design consideration and adequate pitch and end distance L is provided according to:

$$F_p = LF_u/2d \leq 1.5F_u$$

Article 1.14.5 (page 15-1-39):

Revise to read:

1.14.5 Welding Inspector Qualification and Certification

(a) Welding Inspection.

Acceptance of materials and workmanship incorporated in fracture critical members shall be based upon quality control inspections performed by Contractor/Fabricator, Erector personnel and verified by quality assurance inspections performed by representatives of the Engineer. All Inspectors responsible for the QC and QA acceptance or rejection of materials and workmanship shall be qualified as follows:

1. The Inspector shall be a AWS Certified Welding Inspector (CWI) qualified and certified in accordance with the provisions of AWS QC1, Standard For Qualification and Certification of Welding Inspectors, or
2. The Inspector shall be qualified by the Canadian Welding Bureau (CWB) to the requirements of the Canadian Standard Association (CSA) Standard 178.2, Certification of Welding Inspectors, or
3. The Inspector shall be an Engineer or Technician who, by formal training and experience in metals fabrication, inspection and testing, is acceptable to the Engineer as an equivalent to 1. or 2.

Inspection of FCM's shall include all inspection of materials and workmanship prior to welding, during welding and after welding, together with required record keeping, to insure that FCM's are fabricated from accepted materials, by approved processes, stored and shipped in accordance with the requirements of the Contract Documents.

The Inspector may be supported by Assistant Inspectors who may perform specific inspection functions under the supervision of the Inspector. Assistant Inspectors shall be qualified by training and experience to perform the specific functions to which they are assigned. The work of Assistant Inspectors shall be regularly monitored by the Inspector, generally on a daily basis.

The name of the Quality Control Inspector and the Quality Assurance Inspector responsible for each individual fracture critical member shall be listed on all appropriate acceptance forms.

Article 1.14.7 and **Table 1.14.7** (pages 15-1-39 and 15-1-40):

Revise to read:

1.14.7 Notch Toughness of Steel in Fracture Critical Members

(a) Charpy V-notch (CVN) impact test requirements for steels in FCM's shall be as given in Table 1.14.7., except as shown in footnote 6.

TABLE 1.14.7—IMPACT TEST REQUIREMENTS[1] FOR STRUCTURAL STEEL—FRACTURE CRITICAL MEMBERS

ASTM Designation	Thickness, In.	Minimum Average Energy, Ft.-Lb. and Test Temperature		
A 36 or A 709, Grade 36F[5]	Through 1-1/2 Over 1-1/2, Through 4	25 @ 70°F 25 @ 70°F	25 @ 40°F 25 @ 40°F	25 @ 10°F 25 @ -10°F
A 572, Grade 50[2] or A 709, Grade 50F[2, 5]	Through 1-1/2 Mechanically Fastened	25 @ 70°F	25 @ 40°F	25 10°F
or A 588 A 709, Grade 50WF[2, 5]	Over 1-1/2, Through 4 Mechanically Fastened	25 @ 70°F	25 @ 40°F	25 @ -10°F
	Through 1-1/2 Welded	25 @ 70°F	25 @ 40°F	25 @ 10°F
	Over 1-1/2, Through 2 Welded	25 @ 70°F	25 @ 40°F	25 @ -10°F
	Over 2, Through 4 Welded	30 @ 70°F	30 @ 40°F	30 @ -10°F
A 572, Grade 60[3]	Through 1-1/4 Mechanically Fastened	25 @ 70°F	25 @ 40°F	25 @ 10°F
Minimum Service Temperature[4]		0°F	-30°F	-60°F

Revise Footnote 1 and add Footnote 6 as follows:

[1]Impact tests shall be in accordance with the CVN tests as governed by ASTM Designation A 673. Zones 1, 2 and 3 shall require frequency of testing P. In addition, zone 3 shall require impact tests on a set of specimens taken from the other end of the plate.

[6]Steel backing for groove welds joining steels with a minimum specified yield strength of 50 ksi or less may be base metal conforming to ASTM A36, A709, A588, and/or A572, at the Contractor's option, provided the backing material is furnished as bar stock rolled to a size not exceeding 3/8 inch by 1-1/4 inches. The bar stock so furnished need not conform to the Charpy V-Notch impact test rquirements of Table 1.14.7.

Article 1.14.8 (page 15-1-41):

The current paragraphs (h) and (i) will become paragraphs (i) and (j), respectively. The new paragraph (h) will read as follows:

(h) Heats and lots of welding consumables which have been tested and accepted may be interchanged with other tested and accepted heats and lots of welding consumables that are of the same specification made by the same manufacturer.

The following is to be added as a second paragraph to (j), (formerly paragraph (i)):

Welders and welding operators qualified by mechanical and radiographic tests, as required by Article 1.14.8(j), may be requalified annually by radiographic testing of production groove welds acceptable to the Engineer. The Contractor shall maintain a file of radiographs of production welds used to requalify welders or welding operators. The Engineer shall accept evidence of prior qualification provided qualification test records are complete, properly documented, and have been witnessed by a Railroad representative or independent third party for the purpose of implementing this provision.

Article 1.14.9 (page 15-1-42):

Revise paragraphs (a) and (b) to:

(a) Welding Procedure Specifications for joining FCM's are not prequalified. Welding Procedure Specifications shall be qualified by tests as described below. Tests shall be conducted within one year prior to the beginning of welding. Unless otherwise specified in the Contract Documents, the Engineer shall accept properly documented evidence of acceptable qualification testing witnessed by a Railroad representative or an approved independent third party.

(b) The welding procedure tests, as prescribed in Figure 1.14.9, shall be used. The base metal shall be the same ASTM Grade as that to be used in the FCM's except that at option of the Fabricator, ASTM A588 steel may be used to qualify welds in approved base metals which have a minimum specified yield strength of 50 ksi or less. When ASTM A588 test plate and backing steel is used to qualify steels with F_y equal to or less than 50 ksi, the A 588 steel shall have the following minimum chemistry: Carbon .15%; Manganese 1.00%; Silicon .25%; Chromium .50%; and Vanadium .03%.

Article 1.14.9, Notes for Figure 1.14.9 (pages 15-1-43 and 15-1-44)

Revise Note 2 and add Note 8 as follows:

2. All test plate and backing material shall have proper heat identification. Certified mill test reports shall demonstrate that the metal meets the chemical and mechanical requirements specified in 1.14.9.1(b). If ordered by the Engineer, the Charpy V-Notch toughness of the base metal shall meet the requirements of Table 1.14.7.

8. Welding consumables used in this test shall conform to all the requirements of this FCP and shall be of the same specification, grade, and manufacture as the consumables that will be used in the work. Heat and lot testing of welding consumables need not be complete and reported before Welding Procedure Qualification Testing is begun. All requirements of the FCP must be complied with before acceptance of the FCM.

Table 2.2.1A (page 15-2-3):

Revise footnote 4. to:

[4]A 588 and A 709, Grade 50W have atmospheric corrosion resistance of approximately 4 times that of A 36 steel without copper.

Table 2.2.1B (page 15-2-3):

Revise footnote 1. by deleting parts (a) and (b) and rewording to:

[1]Impact tests shall be in accordance with the Charpy V-Notch (CVN) tests as governed by ASTM Specification A 673 with frequency of testing H.

Article 9.1.4 and 9.2.4 (page 15-9-18):

The first two paragraphs on this page (15-9-18) will be revised to read as follows. Note that subparagraphs 3, 4 and 5 of the second paragraph are to be deleted:

The allowable shears in A325 and A490 bolts are based on recommendations of the Research Council on Structural Connections of the Engineering Foundation (1985). Also see Bibliography items (39) (46).

If the Engineer elects to use stresses listed in Table 9.1.4 for special surface treatments, the condition of the contact surfaces shall conform to the following requirements as applicable:

1. Classes A and B (uncoated): Contact surfaces shall be free of oil, paint, lacquer, or other coatings and loose oxide.

2. Class C (hot dip galvanized and roughened):
   Contact surfaces shall be lightly scored by wire brushing or blasting after galvanizing and prior to assembly.

Table 9.1.4 (page 15-9-19):

This table will be replaced with the following:

Table 9.1.4
Allowable Load for Slip-critical Connections
(Slip Load per Unit of Bolt Area, ksi)

Contact Surface of Bolted Parts	Hole Type and Direction of Load Application							
	Any Direction				Transverse		Parallel	
	Standard		Oversize & Short Slot		Long Slots		Long Slots	
	A325	A490	A325	A490	A325	A490	A325	A490
Class A (Slip Coefficient 0.33) Clean mill scale and blast-cleaned surfaces with Class A coatings[a]	17	21	15	18	12	15	10	13
Class B (Slip Coefficient 0.50) Blast-cleaned surfaces and blast-cleaned surfaces with Class B coatings[a]	28	34	24	29	20	24	17	20
Class C (Slip Coefficient 0.40) Hot dip Galvanized and roughened surfaces	22	27	19	23	16	19	14	16

[a]Coatings classified as Class A or Class B includes those coatings which provide a mean slip coefficient not less than 0.33 or 0.50, respectively, as determined by Testing Method to Determine the Slip Coefficient for Coatings Used in Bolted Joints, see Appendix A of Reference 46.

Article 9.1.14.8 (page 15-9-27):

Add the following after the last paragraph of this article:

Steel backing for groove welds using rolled bar stock of limited cross sectional area is considered superior to backing produced by stripping from plate. Bar stock is uniform in cross section and has light mill scale in most instances. Studies of the effects of backing chemistry on weld metal properties indicates that A36 steel is suitable backing for all groove welds in steels with a minimum specified yield stress of 50 ksi or less. The Charpy V-Notch toughness of backing bars of limited dimension will not have a significant influence on the fracture resistance of the groove welds.

It is absolutely essential that all weld backing be continuous and that welds used to join segments of backing be made before the backing is applied to the weld. All joints in backings should be subject to

the same weld quality standards and nondestructive tests specified for similar groove welds in the structure.

It is not essential that each heat and lot of welding consumables be pretested in the combination that will be actually used in the work. Accepted heats and lots of welding consumables that conform to the same specification and are made by the same manufacturer may be interchanged without concern that the weld metal produced will be unacceptable.

Article 9.1.14.9 (page 15-9-29):

After the last paragraph of this article, add the following:

It is intended that welders, welding operators, and tackers be qualified by test within six months prior to the start of fabrication or regularly requalified on an annual basis. Once welders are qualified on the basis of mechanical and radiographic tests, yearly examination of radiographs is considered an acceptable method of assuring that welders and welding operators remain qualified.

It is the intent of these specifications that a fabricator that properly completes welding procedure qualification tests on a yearly basis not be required to repeat the tests for individual railroads unless the railroad has made it a Contract requirement prior to bidding.

The A588 steel test plates and backing specified as an alternative to other steels have been subjected to a metallurgical evaluation that revealed the strength, ductility, and toughness of weld metal produced using this test base metal can be relied upon to indicate whether or not a Welding Procedure Specification will successfully join any of the approved steels with a yield stress of 50 ksi or less. Approval of a single grade of steel will reduce unnecessary testing of base metals and combinations of metals that have no significant effect on the acceptability of the Welding Procedure Specification.

Article 9.3.2.5 (page 15-9-31):

Revise subparagraphs (1), (2) and (3) reworded as follows:

1. *Oversize holes* may have nominal diameters up to 3/16 in. larger than bolts 7/8 in. and less in diameter, 1/4 in. larger than bolts 1 in. in diameter, and 5/16 in. larger than bolts 1-1/8 in. and greater in diameter. They may be used in any or all plies of connections. Hardened washers shall be installed over oversized holes in an outer ply. Where A490 bolts over one inch in diameter are used in oversized holes in external plies, a single hardened washer conforming to ASTM F436, except with a 5/16 in. minimum thickness, shall be used under both the head and the nut in lieu of standard thickness hardened washers. Multiple hardened washers with combined thickness equal to or greater than 5/16 inch do not satisfy this requirement.

2. *Short slotted holes* are nominally 1/16 in. wider than bolt diameter and have a length which does not exceed the oversized diameter provisions for oversize holes by more than 1/16 in. They may be used in any or all plies of connections without regard to direction of loading. Hardened washers shall be installed over short slotted holes in an outer ply. Where A490 bolts over one inch diameter are used in short slotted holes in external plies, a single hardened washer conforming to ASTM F436, except with a 5/16 in. minimum thickness, shall be used under both the head and the nut in lieu of standard thickness hardened washers. Multiple hardened washers with combined thickness equal to or greater than 5/16 inch do not satisfy this requirement.

3. *Long slotted holes* are nominally 1/16 in. wider than the bolt diameter and have a length more than allowed for short slotted holes, but not more than 2.5 times the bolt diameter. The slots may be used without regard to direction of loading. Long slotted holes may be used in only one of the connected parts at an individual faying surface. Where A325 bolts of any diameter or A490 bolts equal to or less than 1 inch in diameter are to be installed and tightened in a long slotted hole in an outer ply, a plate washer or continuous bar of at least 5/16 inch thickness with standard holes shall be provided. These washers or bars shall have a size sufficient to completely cover the slot after installation and shall be of structural grade material, but need not be hardened except as

follows. When A490 bolts over 1 inch in diameter are to be used in long slotted holes in external plies, a single hardened washer conforming to ASTM F436 but with 5/16 inch minimum thickness shall be used in lieu of washers or bars of structural grade material. Multiple hardened washers with combined thickness equal to or greater than 5/16 inch do not satisfy this requirement. If hardened washers are required to satisfy Specification provisions, the hardened washers shall be placed over the outer surface of the plate washer or bar.

Bibliography (page 15-9-43)

Add item #46 as follows:

46. Specification for Structural Joints Using ASTM 325 or A490 Bolts, R.C.S.C., November 13, 1985.

Proposed 1989 Manual Revisions To Chapter 22—Economics of Railway Construction & Maintenance

It is proposed to retitle Part 3, Section 3.2 to "Track Analyses and Design Criteria." The following will replace existing material in Article 3.2.1:

3.2 TRACK ANALYSES AND DESIGN CRITERIA

3.2.1 Design Analyses of Rails and Ties

(a) Foundations of the Design Analyses

Because of the many variables involved in tie and ballast work, the calculation of track deflections and stresses cannot be regarded as exact. However, in-service tests have shown that the track structure can be analyzed within acceptable limits of accuracy by considering the rail as equivalent to a continuous beam resting on an elastic base.[1,2]

The analysis is simplified by the use of the curves shown in Figure 1. This figure shows the influence curves [Appendix I equations (2) and (3) and Ref. 1, First Progress Report, 1918, p. 886] for the vertical rail deflection, the intensity of the "continuous" pressure between rail and ties, and the bending moment in the rail, caused by vehicles at rest.

(b) Determination of Loads to be Used

The analysis should take into consideration the anticipated wheel loads of cars and locomotives and their spacings. The position of the wheels over the influence curves should be such that the analysis yields the maximum values of interest, like bending moments or track deflections.[3] The dynamic effects are taken into consideration by means of the speed effect formula presented below.

(c) Dynamic Effect on Wheel Loads

Maximum stresses in track occur due to dynamic loading and are affected by such factors as train speed, track stiffness, suspension properties of the rolling stock, and track and wheel irregularities. The speed effect coefficient θ recommended by the Association of American Railroads is:[4]

$$\theta = \frac{33 \times \text{Speed (Miles Per Hour)}}{100 \times \text{Diameter of Wheels (Inches)}}$$

Thus, the dynamic wheel load is

$$P^d = (1 + \theta)P$$

where P is the static wheel load.

(d) Rail Support Modulus k

In the suggested analysis (Appendix I) the stiffness of the rail base, known as the elastic modulus of rail support, is denoted by the letter "k." It represents the combined vertical stiffness (yield ability) of the ties, ballast, roadbed, and tie pads (when used), but does not involve the stiffness of the rail. It is defined in lb/in^2 or N/m^2. The value of k is related to the type of ties, tie spacing, tie pad compressibility (when used), depth and quality of ballast, stiffness of subgrade, character of tamping, and generally ranges within the limits of about 1,000 to 3,000 lb/in^2 for one rail. For concrete tie tracks, the k-value will generally be higher. It depends on tie size and its bending stiffness. The k-value during the Winter (frost action) may be three times as large as during the summer. Note, that the k-value for a track is twice the k-value for one rail.

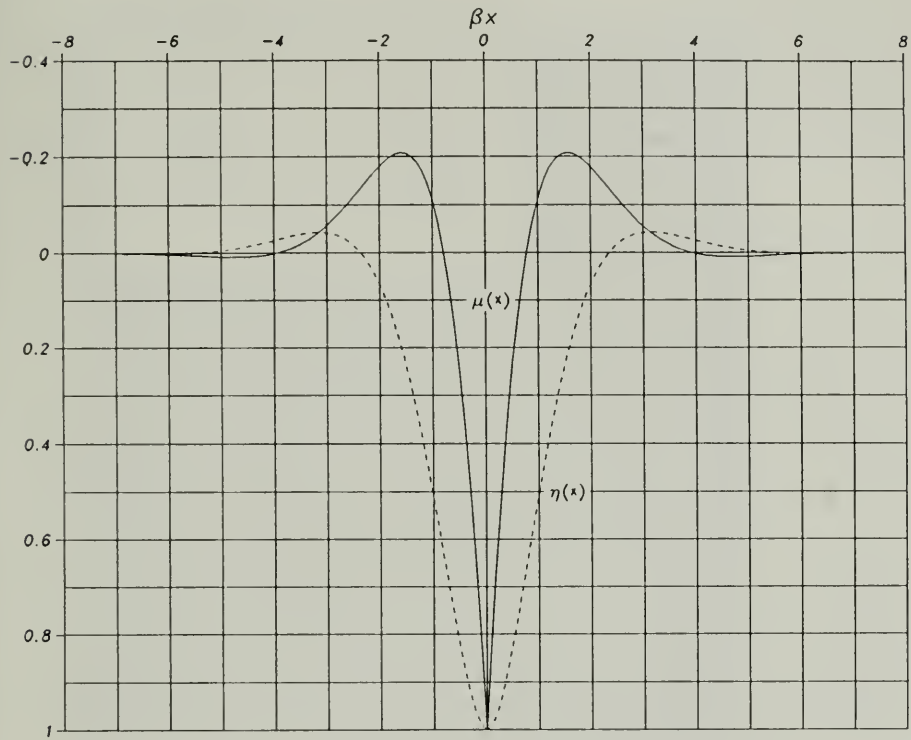


Figure 1

In the past, various methods and special loading devices were used to determine the k-value of a railroad track.[5,6,7] Recently a simple method was proposed for the determination of the k-value in the field by using any available car or locomotive. In this method, a loaded car (or locomotive) is placed on a scale and the wheel loads, P, of one truck are determined. At the location of interest, a measuring stick is attached vertically to the rail web. A level placed about 20 yards (or meters) away from the track is focused on this stick. The car (or locomotive) is then moved at slow speed (about 5 mph) over the track. When the first wheel is directly over the measuring stick, the rail deflection, w_m, is recorded. Next the ratio w_m/P is calculated and the corresponding k-value is determined, for two axle trucks, from the graphs given in Figure 2. More details are given in Ref. 8. For example, for a truck with 119 RE rails, a two axle truck with wheel loads P = 30,000 lb. = 15 tons caused a deflection under one of the wheels w_m = 0.15 inches. Then w_m/P = 0.15/15 = 0.01 in/ton and according to Figure 2, k = 2,800 lb/in^2.

(e) Tie Pressure on Ballast

The average intensity of tie pressure on ballast should not exceed 65 psi for good track conditions.[9] In-service tests have determined that the load on an individual tie can be as high as 2.7 times the normal value computed from the elastic theory and load increases of 60% were frequent and fairly general. This is due to a variety of reasons related to the variation in track condition and the method of analysis.[10] In order to account for this increase in loading, it is suggested that the calculated rail seat loads be doubled when using the analysis presented in Appendix I.

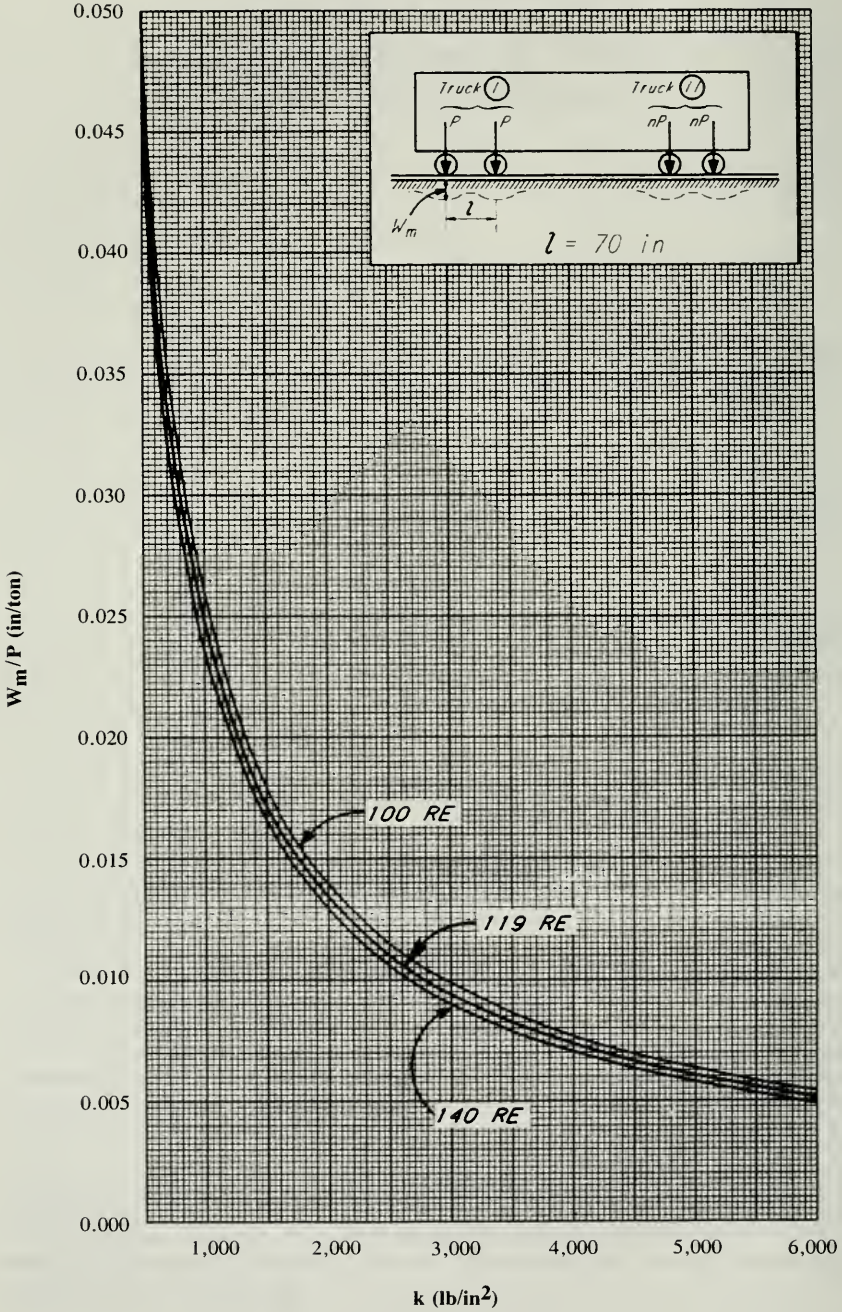

0.050
0.045
0.040
0.035
0.030
0.025
0.020
0.015
0.010
0.005
0.000
W_m/P (in/ton)
1,000
2,000
3,000
4,000
5,000
6,000
k (lb/in^2)
Truck I
Truck II
P
P
nP
nP
W_m
l
l = 70 in
100 RE
119 RE
140 RE

Figure 2

(f) Pressure Between Ballast and Subgrade

One of the functions of ballast is to transmit the tie load to the roadbed with diminished unit pressure. The pressure on the subgrade is a function of the depth of ballast. It is recommended that the calculated pressure on subgrade be smaller than 25 psi for good subgrade conditions. This value should be reduced for subgrades of poor quality. Analytical determination of ballast depth is covered elsewhere.

(g) Allowable Bending Stress in the Rail

According to the Association of American Railroads, "the acceptable rail stress for continuous welded rail" is established as follows:

Reduction due to:	Lateral Bending	20%
	Track Condition	25%
	Rail Wear and Corrosion	15%
	Unbalanced Elevation	15%
	Temperature Stress	20,000 psi

Assuming the yield strength of rail steel is 70,000 psi, the largest calculated rail stress at the base due to flexure should be smaller than:

$$\sigma \text{ all} = \frac{70{,}000 - 20{,}000}{1.20 \times 1.25 \times 1.15 \times 1.15} = 25{,}000 \text{ psi}$$

(h) Allowable Track Deflections

The up and down motion of a ballasted track caused by the passing wheels of moving trains results in the deterioration, with time, of the line and surface of the track structure. Therefore, a softer track that deflects more readily than a stiffer track requires more maintenance. In order to maintain good ride quality and to reduce maintenance costs, it is recommended that vertical rail deflections be limited to 1/4 inch.[12]

REFERENCES

1. A. N. Talbot: "Stresses in Railroad Track," Progress Reports 1-7, AREA Proceedings, 1918-1940. Reproduced by AREA in 1980 as "Stresses in Railroad Track - The Talbot Reports."
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6. A. M. Zarembski and J. Choros: "On the Measurement and Calculation of Vertical Track Modulus" Proceedings AREA, Vol. 81, 1981.
7. A. D. Kerr and H. W. Shenton, III: "On the Reduced Area Method for Calculating the Vertical Track Modulus," Proceedings AREA, Vol. 86, 1985.
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9. Proceedings AREA, Vol. 67, p. 622.
10. A. D. Kerr and H. W. Shenton, III: "Railroad Track Analyses and Determination of Parameters," Journal of Engineering Mechanics, ASCE, Vol. 112, No. 11, 1986.
11. G. M. Magee: "Calculation of Rail Bending Stresses for 125 ton Tank Car," AAR Research Center, Report No. 19506, April, 1965, Chicago.
12. Proceedings AREA, Vol. 35, p. 1149.

APPENDIX I

Details of Track Calculations

The analysis is conducted by considering the rail to be a continuously supported beam on an elastic foundation consisting of closely spaced springs.[1]

1. The recommended differential equation for rail deflections is:

$$EI \frac{d^4w}{dx^4} + kw = q(x)$$

where:

E = Young's modulus of rail steel
I = moment of inertia of one rail with respect to the horizontal centroidal axis
w = vertical track deflection
q = vertical load distribution (wheel loads) on one rail
x = point on rail axis
k = elastic modulus of rail support, for one rail
kw = p = "distributed" rail-tie contact pressure

The modulus of rail support, k, was discussed above. Note that the value of k is not affected by the rail properties.

By solving Equation (1), for one wheel load P, the magnitude of the deflection, w, at point x of track and bending moment, M, can be expressed by the following expressions:

$$w(x) = \frac{\beta P^d}{2k} \underbrace{e^{-\beta x}(\cos \beta x + \sin \beta x)}_{\eta(\beta x)} \quad (2)$$

$$M(x) = \frac{P^d}{4\beta} \underbrace{e^{-\beta x}(\cos \beta x - \sin \beta x)}_{\mu(\beta x)} \quad (3)$$

where $P^d = (1 + \theta)P$ = dynamic wheel load and

$$\beta = \sqrt[4]{\frac{k}{4EI}} = \text{stiffness ratio} \quad (4)$$

$\mu(\beta x)$ and $\eta(\beta x)$ are shown in Figure 1.

2. Dynamic Loading for Determination of Track Stresses

Maximum stresses in track occur under dynamic loading. As discussed in Section 3.2(c), the speed effect coefficient θ recommended by the Association of American Railroads for taking into consideration dynamic effects is:

$\theta = 33v/(100D)$ where v = dominant train speed (mph) and D = diameter of vehicle wheels (inches).

Example: For a maximum speed of 75 mph and wheel diameter of 36 inches $\theta = 33.75/3600 = 0.7$. For a static wheel load P = 30,000 lb., the dynamic wheel load for design purposes is $P^d = 1.7P = 51{,}000$ lb.

3. For one wheel load, the maximum rail deflection, w_{max}, and maximum rail bending moment, M_{max}, take place at the wheel. They are obtained by setting x = O in Equations (2) and (3).

$$w_{max} = w(0) = \frac{\beta P^d}{2k} \tag{5}$$

$$M_{max} = M(0) = \frac{P^d}{4\beta} \tag{6}$$

For the closely spaced axles of a truck, the effect of all wheel loads in the vicinity have to be considered on w_{max} and M_{max}, using the influence curves shown in Figure 1. Examples of such analyses are presented in Ref. 3. Note that for a two axle truck, the largest rail deflection does not usually occur under a wheel.

4. For design purposes, the maximum dynamic bending stress in the rail is

$$\sigma^d_{max} = \frac{M^d_{max} \cdot c}{I} = \frac{M^d_{max}}{Z_b} \tag{7}$$

Note that the stresses caused by the axial force were taken into consideration in σ all, as shown in Section 3.2(g). In Equation (7), c = distance from neutral axis to rail base, Z_b = section modulus for rail base (listed in AREA Manual for Railway Engineering, Chapter 4 Part 1), and M^d_{max} = dynamic maximum bending moment due to all wheel loads.

Using the design criterion $\sigma^d_{max} \leqq \sigma allow = 25{,}000$ psi, as given in Section 3.2(g), it follows that $M^d_{max}/Z_b < \sigma all$. Thus, the required section modulus is

$$Z_{req} > \frac{M^d_{max}}{25{,}000} \tag{8}$$

This is a method for choosing the rail section.

5. The intensity of the "continuously" distributed pressure p against the underside of the rail is given by:

$$p(x) = kw(x) \tag{9}$$

The influence curve for w (η in Figure 1) is used to determine the largest value p_{max}, caused by more than one wheel load. The maximum rail seat load F_{max} on an individual tie is given by:

$$F_{max} = p_{max} \times a \tag{10}$$

Where F = rail seat load (pounds)
p = rail base pressure (pounds/inch)
a = tie spacing (inches)

6. Tie plate size is determined from condition

$$\frac{F^{d}_{max}}{A} < \sigma_{all.wood} \qquad (11)$$

Thus, the required tie-plate area is

$$A_{req} > \frac{F^{d}_{max}}{\sigma_{all.wood}} \qquad (12)$$

where $\sigma_{all.wood}$ is the allowable contact stress between tie plate and tie. Tests have shown that for hardwood ties this stress is 400 psi and for softwood ties 250 psi (W. W. Hay, "Railroad Engineering," 2nd Ed., p. 576).

7. Next determine the effective bearing area of a tie and allowable bearing pressure. The contact pressure between tie and ballast for a well maintained track is largest at the rail seat and smallest at the tie center. This pressure distribution varies with accumulated traffic and track condition. In order to simplify the calculations, the dashed uniform distribution may be assumed, with

$$L_{eff} \cong L/3 \qquad (13)$$

as shown in Figure 3.

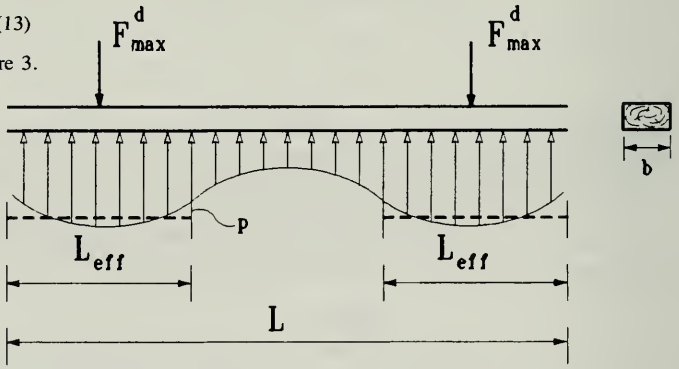

Figure 3

Then the effective bearing area of the tie is

$$A_b = b \times L_{eff} = \frac{1}{3}(b \times L) \qquad (14)$$

where b = width of tie at base. The corresponding tie-ballast bearing pressure is

$$\frac{3 \times F_{max}}{b \times L} < 65 \text{ psi} \qquad (15)$$

For detailed track analyses, which are based on the method described above, check with Reference 3 listed previously.

Proposed 1989 Manual Revisions To Chapter 27 — Maintenance of Way Work Equipment

The "Specifications For On-Track Roadway Machines" in Part 2 has been reviewed with changes proposed to various paragraphs that will be rewritten as follows:

Article 1, Paragraph 1.3 (page 27-2-22):

1.3 Reliability: Design, construction and materials used in the equipment shall assure that it will function reliably and efficiently in sustained operation under hard usage in an adverse railway environment, including but not limited to severe grades and superelevation.

Article 2, Paragraph 2.2 (page 27-2-23):

2.2 All fasteners shall meet strength requirements of ASTM-A449 or stronger. All bolted applications shall have at least two full threads protruding beyond the nut after the fastener has been properly torqued.

Article 3, Paragraph 3.5, 3.6 and 3.7 (pages 27-2-23 and 24):

3.5 In emergency situations a system must be provided for releasing each brake in not more than 2-1/2 minutes per wheel.

3.6 Air brake systems must utilize SAE J1402 Table A (formerly Type E), DOT #FMV-SS 106-74 Type A1 air brake hose with reusable fittings. Brake system must have pressure regulator, pressure gauge and standard truck type reservoir.
Reservoir must conform with SAE J10-B specifications. Manufacturers shall also be able to provide these types of reservoirs which meet ASME specifications for certification when requested as an option. System must be activated by standard truck type, foot or hand operated, control valve and one truck type, quick release valve for each two wheels.

3.7 Air brake system must maintain operating pressure, above 70 psi but not more than 105 psi. An audible low pressure alarm shall be furnished which will activate whenever the pressure is 70 psi or less.

Article 4, Paragraph 4.2 (page 27-2-24):

4.2 Wheel contour and gage shall comply with standards set by AREA (Engineering Division, AAR) or Mechanical Division, AAR as required for size and type of wheels furnished.

Article 8, Paragraph 8.2 (page 27-2-25.2)

8.2 Machines equipped with enclosed cab or cab top shall have exhaust discharge at least 12 inches above cab top.

Article 9, Paragraph 9.3 (page 27-2-25.2)

9.3 Manufacturers must pre-filter all oil through a 10-micron absolute or finer filtering system on initial filling of hydraulic system.

Article 9, Paragraph 9.4, items a.3. and a.4. (page 27-2-25.3)

a.3. An air inlet breather which is of sufficient capacity to maintain approximate atmospheric pressure at maximum demands on the hydraulic system and to assure vacuum at pump inlet(s) shall not exceed 60 percent of pump manufacturer's recommendations. Air breather system shall be equipped with a 10-micron ($B_{10} = 10$) or finer filter, either cartridge or spin-on type.

EXCEPTION: Sealed and pressurized system.

a.4. A filler with at least a 100-mesh screen, protected from external damage. Filler shall have a minimum capacity of 5 gallons per minute with 5,000 ssu viscosity fluid. Filler cap shall have a retainer that can be locked with a large padlock, similar to the type stated in 7.2.

Manufacturer shall provide, as an option, a system to fill reservoirs by use of an external hand or power hydraulic pump through a separate 10-micron ($B_{10} = 10$), screw on, non-bypass filter. The filtration capacity of the filter assembly shall be rated and designed to accommodate 30 GPM at 32° F., based on manufacturer's recommended hydraulic oil. This system should also contain a sealed hydraulic system with a 10-micron ($B_{10} = 10$) breather and elimination of the normally supplied fill cap with strainer. A 1" NPT 'T' shall be provided between the separate hydraulic filter and replenishing pump so the system can easily be set up for filling from a service vehicle.

Article 9, Paragraph 9.7 (page 27-2-25.4)

9.7 Pressure testing tee(s) shall be provided at locations to provide easy access for checking hydraulic pressures on all circuits. The tee(s) shall include a 1/4" NPT fitting with a male quick-disconnect fluid coupling.

Article 9, Paragraph 9.10a., b. and c. item 3 (pages 27-2-25.4 and 27-2-25.5)

a. Fluid conductors utilized in circuits operating under 3000 psi must use high pressure hose; SAE 100 R2 Type A, Hi-Impulse type with the following qualification requirements:

b. All fluid conductors utilized in circuits operating over 3000 psi or in hydrostatic drive systems should use extra high pressure hose; SAE 100 R12 type when utilizing hose sizes through 1 inch, which have the following qualification requirements.

c.3. Tested to 500,000 impulse cycles at 250 degrees F.

Article 10, Paragraph 10.11.d. (page 27-2-25.7)

d. Only pipes as electrical conduit or pneumatic tubing as used exclusively for control circuitry shall enter cabinet.

Article 17, First Paragraph (page 27-2-25.10)

17. **PAINT**

Manufacturer shall use paints which do not require periodic medical examinations or special equipment, other than passive respirators. It is assumed that no-lead paint with a lower percentage of solvent may be required and that exact colors formerly furnished may not be available. Manufacturer shall provide the brightest or most easily identifiable colors available.

Proposed 1989 Manual Revisions To Chapter 28 — Clearances

A proposed "Recommended Procedure to Insure that Additions or Modifications to Clearances are Promptly and Properly Reported to Clearance Engineer" has been developed for Chapter 28. The proposed procedure is as follows:

3.10 Recommended Procedure to Insure that Additions or Modifications to Clearances are Promptly and Properly Reported to Clearance Engineer

Purpose

To establish a recommended standard procedure for the Railroad Industry to follow which can be disseminated to all concerned personnel in the form of "guidelines" or "instructions" *to help raise a higher level of awareness* regarding problems associated with reporting planned and unplanned modifications to the clearance file maintained by the Clearance Engineer or designated person responsible for maintaining clearance data.

Procedure

I. Existing Obstructions

A. **Limiting Obstructions:**

It is recommended that a list of the most restrictive overhead and lateral clearance obstructions be furnished to designated key field officers so that they may be made aware of controlling clearance locations on their respective territories. This can be furnished best by:

1. Listing information in tabular form by line segments and/or providing
2. Copies of individual or composite obstruction diagrams.

B. **Modifications to Obstructions:**

1. Planned modifications.
   a. Clearance Reduction-provide Clearance Engineer with letter indicating approval by officer having authority in such matters.
   b. Reporting Modifications-
      (1) Mandatory if modification either reduces or improves clearance.
      (2) Falsework-
         a. Clearance Engineer must be consulted prior to any temporary clearance authorization.
         b. Clearance Engineer protect temporary clearance, recording location, type of modification, dimensions and date of approval.
         c. Designated field engineer or supervisory officer advise Clearance Engineer of temporary clearance measurements on erection date and also furnish new dimensions to modified obstruction promptly upon removal of temporary clearance.
      (3) Other Modifications-follow steps (2) (a), (b) and (c).
2. Modifications due to MW&S Activities:
   a. Examples of common occurences that require remeasurement and reporting due to MW&S related activities that could affect clearances are:
      (1) Laying heavier rail - affects clearances to overhead bridges, thru truss bridges and tunnels.

(2) T&S and Surfacing Operations -

(i) Track normally raised - Clearance Engineer to advise all concerned, by milepost location, of overhead obstructions having less than 22'-0' vertical clearance, as reminder, prior to track being worked. Company policy, procedure or instructions should predetermine any permissible amount of track raise for specified field conditions.

(ii) Track centers on curves affected by change in superelevation and alignment.

(iii) Lateral clearance to obstructions affected by changes in: superelevation, alignment and temperature effect on welded rail.

(3) Replacement of track in tunnels and under O.H. Bridges.

b. Derailments.

c. Slides.

d. Washouts.

e. Bridge deck replacements.

C. **Improvement to Clearances:**

Limiting clearances can be improved by one or combination of:

1. Track lowering.
2. Raising O.H. Bridge or replacement of components.
3. Track realignment.
4. Change in superelevation.
5. Trimming or removal of rock cuts.
6. Relocation of lateral obstructions such as signals, signs, train sheds, hand rails, etc.
7. Roadbed stabilization.
8. Rebuilding or modifying the obstruction.
9. Complete removal of the obstruction.

D. **Field Measuring and Reporting:**

1. Clearance Engineer should disseminate to key field personnel a pre-printed format, including simple instructions and applicable cross sections with blank dimension lines, for recording new measurements.
   a. This provides record verifying new measurements indicating date measurements taken and by whom.
   b. To measure irregular obstructions such as rock cuts and tunnels, clearance car should be scheduled (See article 3.b.below.)
2. Responsible Division officer or designated representative to measure, record and forward copy of new clearance data to Clearance Engineer.
   a. Track center data to be furnished within time frame prescribed by company policy or outstanding instructions.
   b. Clearance reduction to an obstruction to be reported immediately.
   c. Measurements to be reported to accuracy of 1/8 inch.
3. Use of Clearance Measuring Vehicle.
   a. Frequency of clearance car operation and priority order of line segments requiring remeasurement of clearances should be established by Company policy or outstanding instructions.
   b. Irregular or continuous obstructions such as tunnels, rock cuts and series of thru trusses can be measured more accurately and efficiently by clearance car.
4. Clearance Engineer's address and phone number are:

_____

_____

_____

II. New Construction

A. **Grade Separation**

1. Plans must be reviewed by Grade Separation Engineer or equivalent.
2. Clearances must conform to Railroad's Standard Plans.
3. Temporary or permanent detour tracks must conform to Railroad's Standards and policy or guidelines covering such tracks.
4. Railroad bridges, either temporary or permanent, must conform to Railroad's Standards.
5. Clearance Engineer must be advised of proposed grade separation prior to execution of agreement covering separation.
6. Responsible Division officer or his designated representative must provide Clearance Engineer with minimum vertical and lateral clearance upon completion of separation.
7. Clearance Engineer will update clearance file for the new obstruction.

B. **Temporary Clearances**

1. Railroad Review of Plans:
   a. Grade Separation Engineer (or equivalent) must consult with Clearance Engineer to determine if proposed clearance for falsework is acceptable.
   b. Clearance Engineer will record milepost location, approved temporary clearance and handle to protect.
2. During Construction:
   a. Responsible Division officer or his designated representative must inform Clearance Engineer by phone giving physical measurement to temporary clearance on date of construction.
   b. Clearance Engineer will check to determine if clearance is within acceptable limits. If not, steps must be taken to protect train operations.
   c. Responsible Division officer or his designated representative must advise Clearance Engineer immediately upon removal of temporary clearance.
   d. Clearance Engineer will update clearance file accordingly.

C. **Track Construction**:

1. Prior to construction, engineer or designated officer handling new track construction or retirements must furnish Clearance Engineer with plan showing details of approved projects.
2. Division Engineer or his designated representatives must advise Clearance Engineer of following:
   a. Before work trains or work equipment are operated on new trackage, Clearance Engineer must be given advance notification to protect movement of wide loads.
   b. Date trackage is to be placed in service.
   c. Verification that track centers, alignment and superelevations conform to project plans; otherwise give deviation to plans.
   d. Measurements to any overhead or side obstructions.

D. **Other Types of Construction:**

1. In addition to grade separations and track construction, all other types of new construction which could affect clearances also need to be reported to Clearance Engineer.

   Most common examples are:
   a. Retaining walls.
   b. Railroad signals and signal bridges.
   c. Grade crossing signals/crossbucks.

d. Roadway signs.
e. Communications facilities.
f. Poles and crossarms.
g. Overhead wire lines.
h. Tunnels.
i. Thru type railroad bridges.
j. Buildings.
k. Loading docks with overhanging roofs.
l. Passenger sheds.
m. Passenger platforms.
n. Fences.
o. Handrails.
p. Barricades.
q. Coal loading tipples and grain elevator loading chutes.
r. Canopies.
s. Slide protection fences.

2. Responsibility for reporting.
   a. Division or other personnel must report new fixed obstructions to Clearance Engineer.
   b. Employees patrolling track on scheduled basis must report all new obstructions to their immediate supervisor for further handling as required.
   c. Signal, Communication and Mechanical Departments should also report details of all new obstructions placed on operating right of way that could possibly affect clearances.

3. Updating Clearance File:
   1. Clearance Engineer is to review information furnished from field officers and update records.
   2. If obstruction is substandard, Clearance Engineer will report findings to responsible department for appropriate action.

Proposed 1989 Portfolio Revisions

Six ballots were approved by Committee 5 proposing the following modifications to Portfolio plans:

Revision of AREA Plans 502-71 and 504-71 Guard Rail Setting and Guard Rails

This revision covers longer guard rails with the guard rail flare opposite the frog flare. This revision also covers lengthening the guard rail flare and changing the double cut flare to a single cut flare 3′0″ long.

Revision to the following AREA plans. This revision covers lengthening the flares on rail bound frogs to 3′0″ long.

Plans affected: AREA Plan: 616-88 621-86
617-88 622-86
618-88 623-86
619-88 624-86
620-88 625-88

This revision refers to adding the following note to Plan 504-71.

"Boltless guard rails approved by the purchaser may be used in place of the bolted guard rail design."

Revision to AREA Plan 1010-84 and Manual Section M-2 Manganese Castings.

Add Note as follows to AREA Plan 1010-84.

Note 2: Depth Hardening Specifications, Brinell Hardness Number and Test Locations per AREA Specification 100-82, Section 15 Material Specifications and details, Part M2 Manganese Steel Castings paragraph M2.7 Depth Hardening.

Revise Manual Section M2.7.1, Depth Hardening, by removing processes.

Revised paragraph is as follows:

M2.7 Depth Hardening

M2.7.1 When specified, the impact areas described on the track work plans shall be hardened to a minimum surface Brinell hardness number of 352 for the areas within 1" (25.4 mm) of the gage of guard lines and minimum Brinell hardness number 321 for the balance of the hardened area.

PLAN 1010-85

On the proposed revision to AREA Plan 1010-85, Permissible Variations in Completed Frogs, change the maximum distance that a bolt can protrude beyond the face of the nut from 11/16" to 1-1/16"."

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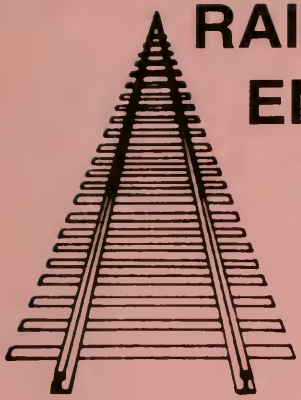

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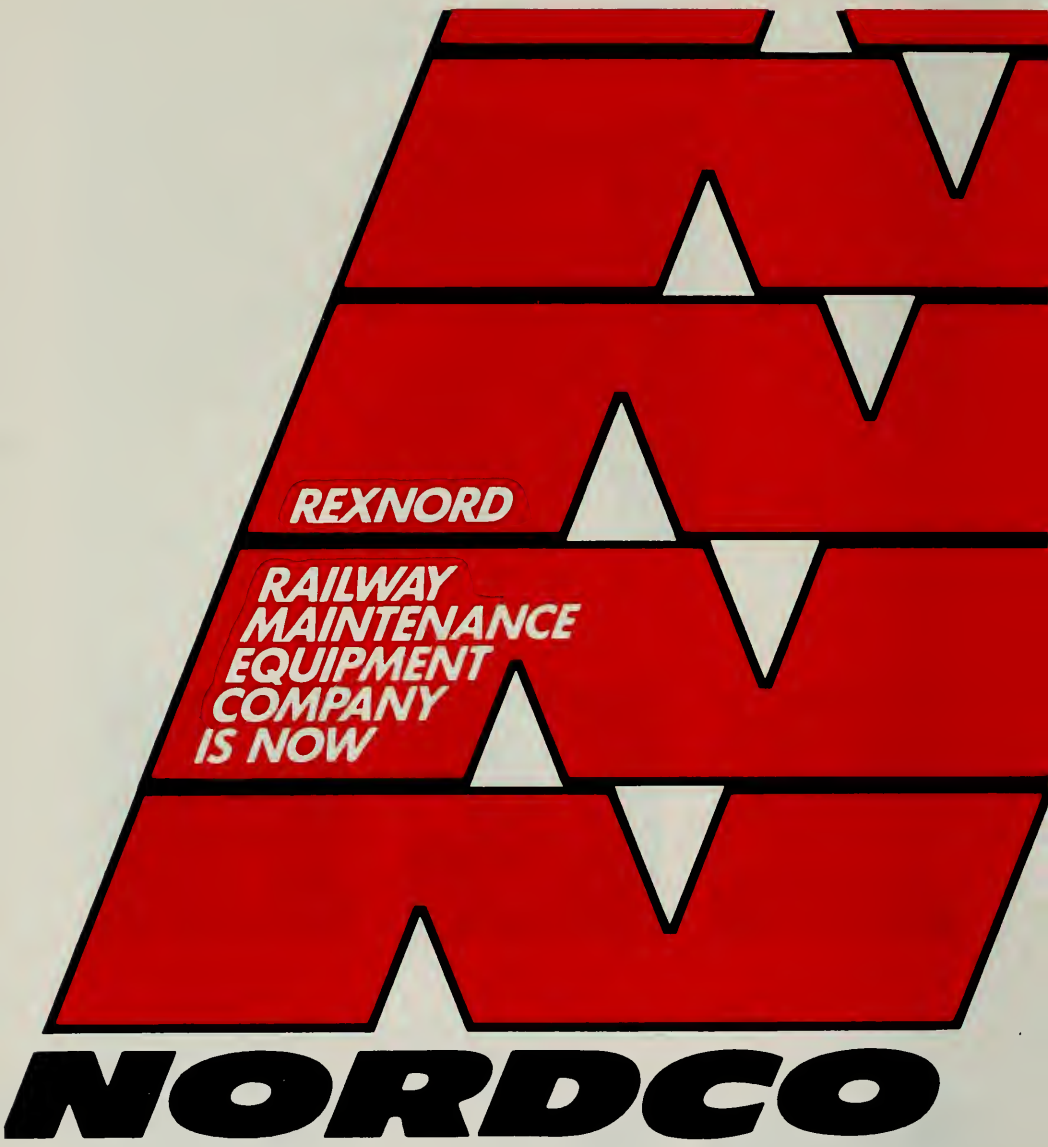

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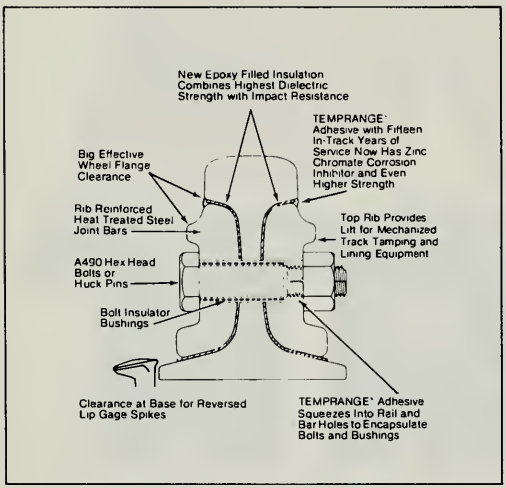
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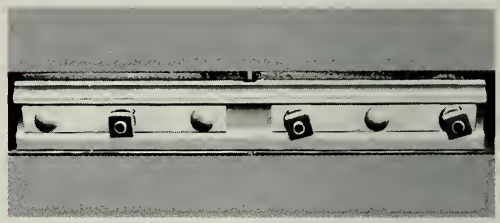

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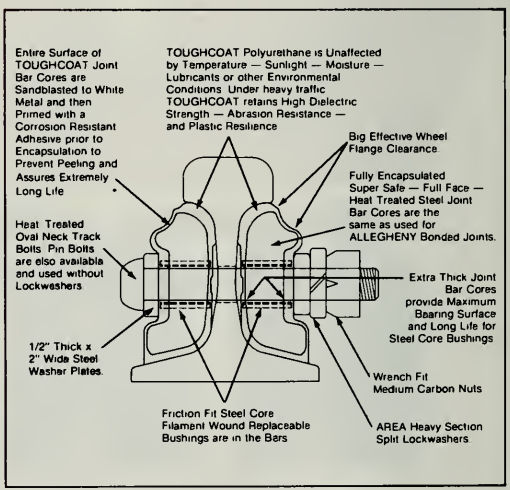
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BULLETIN

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Cover Photo: Northbound empty Alaska Railroad coal train descends 3% grade rounding 14°30′ curve beneath Bartlett Glacier, at milepost 48 between Portage and Moose Pass August 24, 1988.

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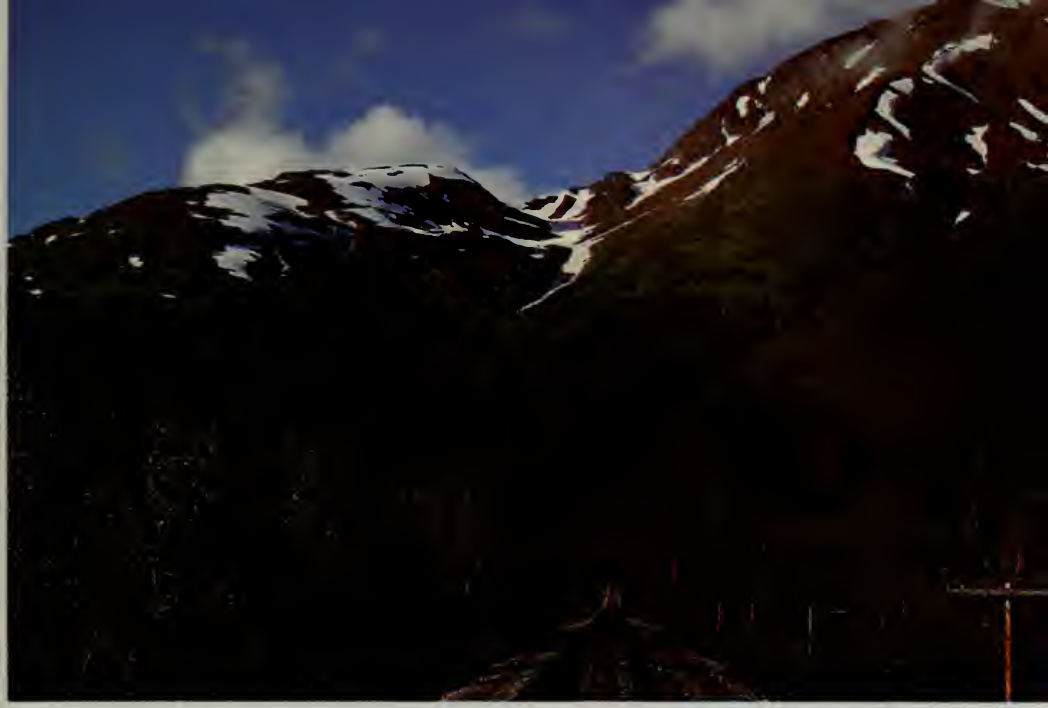


Photo 1—Heading south from Portage into Mountains of Kenai Peninsula.

Alaska—South From Anchorage

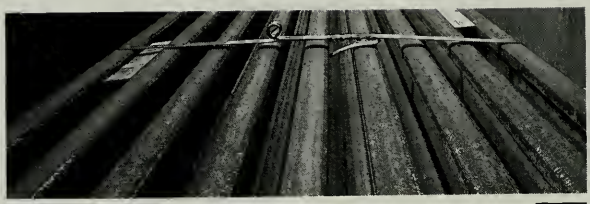
The Alaska Railroad is headquartered in Alaska's largest city, Anchorage, which, with about 250,000 people, contains about half of Alaska's population. The Alaska railroad's most famous line is from here north to Fairbanks, which passes near Mt. McKinley, North America's tallest mountain at 20,320 ft., located in Denali National Park. Some of the world's largest and most luxurious passenger equipment operates on this route.

Less publicized, but still spectacular, if not more so, is the 114-mile line south of Anchorage to Seward, including the 12 mile branch from Portage to Whittier (see map on this page). This area receives much more snowfall than the line to the north because of its proximity to the ocean and has many more glaciers for this reason. The following is a description of these lines based on an inspection trip August 24, 1988.

Anchorage-Seward

This line to the south leaves the Alaska Railroad's Yard and Headquarters office at the base of a hill on which downtown Anchorage is located and follows the shore of the Knik Arm of the Cook Inlet around the west side of downtown Anchorage before heading southeasterly across a relatively flat peninsula on which are located the south suburbs of Anchorage. This flat area emphatically ends where mountains come right to the shores of Turnagain Arm of Cook Inlet.

The very scenic Turnagain Arm would be called a fiord in Norway, and there are high mountains on both shores of this narrow arm of the ocean. The single track railroad, and the two-lane Alaska highway 1, follow closely along the twisting north shore of Turnagain Arm with the mountains and



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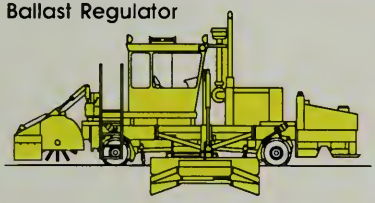

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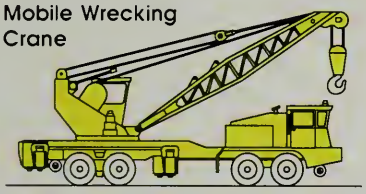

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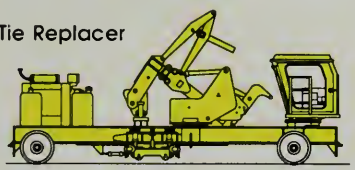

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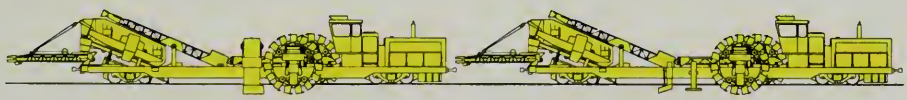

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Photo 2—Northbound empty Alaska Railroad coal train descends 3% grade rounding 14°30′ curve beneath Bartlett Glacier, at milepost 48 between Portage and Moose Pass August 24, 1988.

rocky shores almost continually in view (see photo 6). With an arctic-like vista of glaciers ahead, the line comes to the junction with the Whittier Branch (covered further on in this feature) at Portage, at the head of Turnagain Arm. Here the line turns south, heading up a mile-wide valley into the mountains of the Kenai Peninsula. Highway 1 uses a different route, leaving the railroad as the only sign of human activity in this valley (see photo 1).

The railroad starts climbing the west side of the valley, with beautiful views of the Spencer Glacier to the east. The valley then suddenly becomes very narrow and rocky, with the railroad sharply curving, clinging to rock ledges and cutting through bends in this canyon with tunnels. Some former

tunnels have been daylighted to cure instability problems. The valley widens following this narrow area and becomes steeper where the Bartlett Glacier intrudes from the east. The grade steepens to 3%, and a 14°30′ horseshoe curve occurs directly under the face of the glacier (see photo 2 and cover). This is followed by more curves, which bring the line to the summit at Grandview, where there is a siding to allow heavy southbound freights to double this grade. Shortly beyond Grandview is a view of the Trail Glacier, after which the line descends through the wild meadows and swamps of spectacularly scenic mountain valleys. Approaching Moose Pass, which is not a summit on the railroad, the mountains can frequently be seen reflected in a lake whose shore the tracks follow. Beyond Moose Pass, where Alaska highway No. 1 rejoins the railroad, the line goes down another very scenic mountain valley to the port

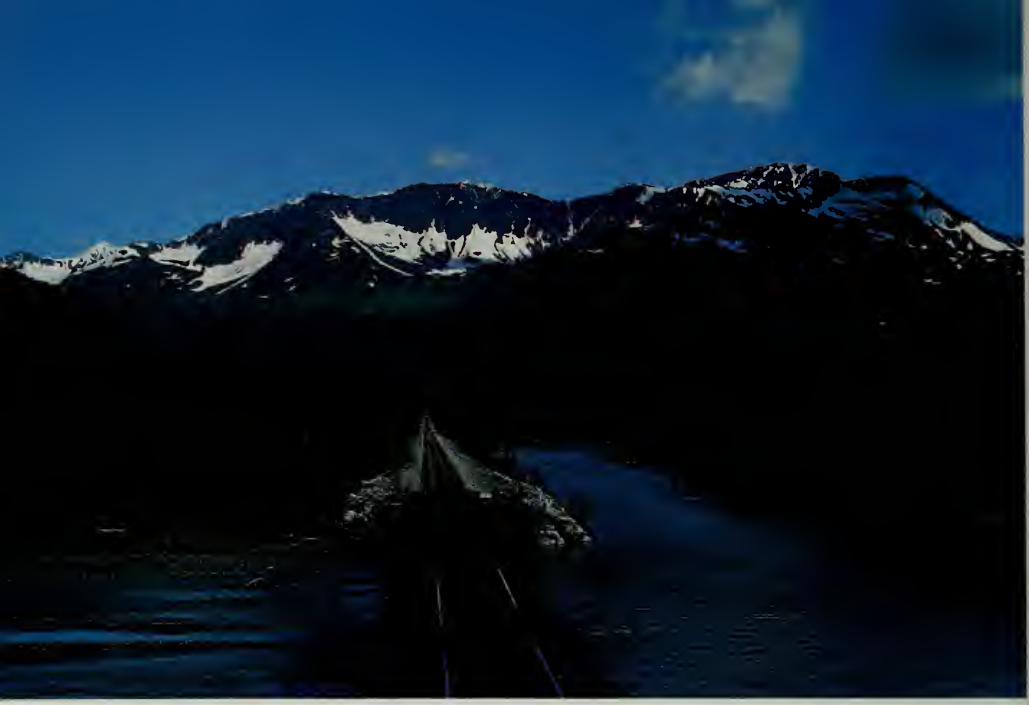

Photo 3—Looking north from north of Moose Pass on Kenai Peninsula.

of Seward on the shores of Resurrection Bay, where there is a yard and port. Export coal for Korea is transferred from train to ship here.

The Whittier Branch

The 12-mile branch from Portage to Whittier heads east up a short valley. The glaciers and snow above the swampy, flat valley floor makes it hard to believe this is August, when the snow cover is at its

Photo 4—Shuttle train (seen here at Portage) carries miscellaneous road traffic because of there being no highway to Whittier.

Photo 5—Whittier Yard on slack day between arrival of railcar-carrying ships from U.S. and Canada.

minimum. The line goes into one tunnel cut through a rock face that protrudes into the stream in the valley, which comes from melting glaciers. The railroad emerges to cross a side valley and its stream for a short interval before heading into the 2-1/2 mile main tunnel under a high, glacier-topped ridge. Both tunnels have power doors at both ends used in winter to prevent ice formation. The line emerges from the main tunnel onto the floor of the deep mountainous valley in which the Port of Whittier is located (see photo 5). Here is where rail cars from the U.S. and Canada arrive by barge and ship and

Photo 6—Watching for Beluga whales along north shore of Turnagain Arm. Alaska Railroad is in foreground.

where most cruise ships and passenger ferries arrive in the area. There are also a pleasure boat marina and old military facilities.

Because there is no highway to Whittier, the railroad runs a "ferry" train of flat cars shuttling from here to Portage. On this day the flat cars on the train had trucks, autos, campers, pick-ups, tour buses, a motorcycle, and even a trailer with a boat (see photo 4). Occupants remained in their vehicles for the trips, including through the unlit tunnel.

This inspection trip certainly confirmed that the 126 miles of railroad line south of Anchorage are among the most spectacular on the North American continent.

The AREA wishes to thank F. C. "Obie" Weeks, Chief Engineer of the Alaska Railroad for the numerous courtesies extended which made this article and its photographs possible.

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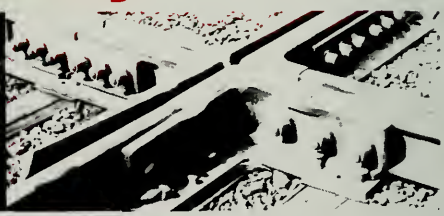

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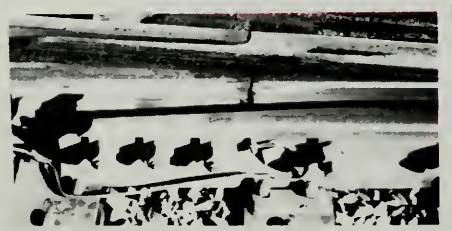

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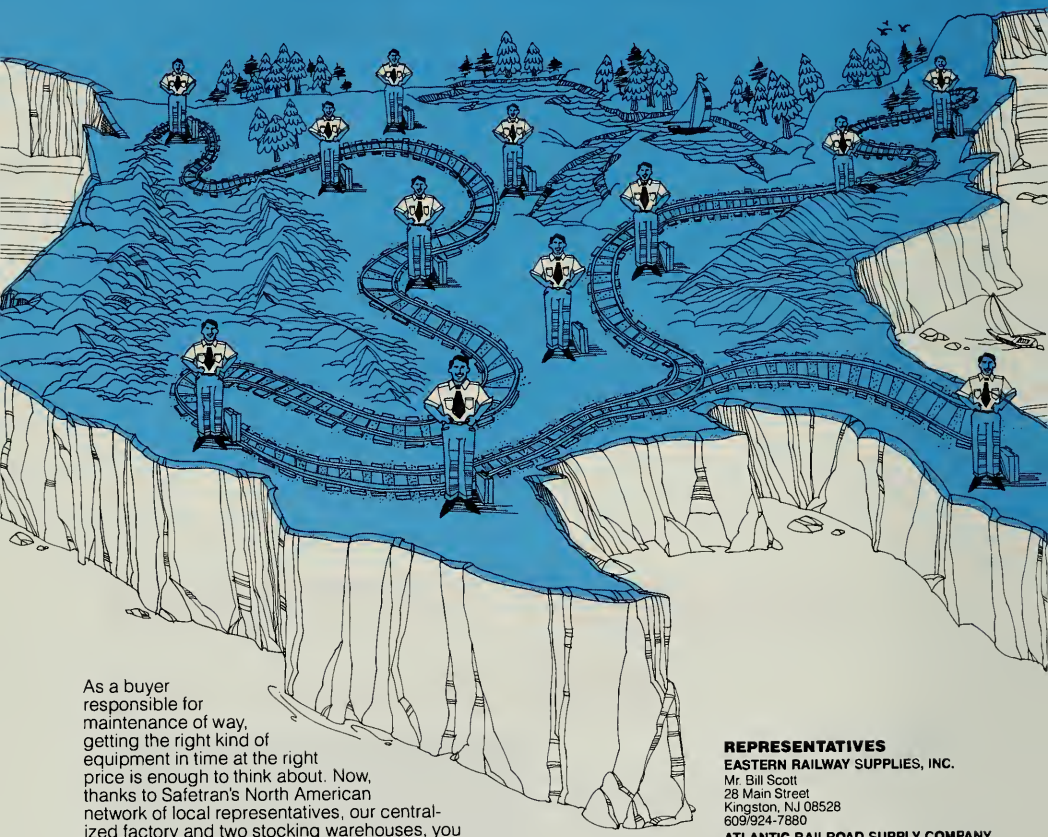

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On the Development of a Computer Model for the Economic Analysis of Alternate Tie/Fastener Configurations

By: Dr. Allan M. Zarembski, P.E.*
Mr. Joseph T. A. Masih, M.C.E.**

ACKNOWLEDGMENT:

This paper describes the development of an economic benefit analysis model. This activity was funded by and performed under the auspices of the Railway Tie Association.*** Specific thanks are extended to Mr. Alan Miller, Mr. Raymond Wingard and Mr. D. B. Mabry for their support and assistance during this activity.

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b. Economic criterion. Is the cost of the component economically viable with other similar components or products?

This second criterion comes into play only if the first is satisfied, i.e. the component or system must have adequate performance, before its relative economics is even considered. Only when that performance criterion is met, does the economic criterion come into play.

An example of this two step process is that of the crossties and fastener systems. While concrete tie designs had been introduced as early as 1893 (1), it has only been in the last decade that their performance has been deemed to be adequate to withstand the severe loading environment of North American freight operations. With the introduction of an American Railway Engineering Association specification (2) in that period and the successful installation of several large scale test sites in severe loading environments, the issue of "adequate" performance of concrete ties appeared to have been met.

At this point, the economics of the alternate tie and fastener systems entered into the consideration of railway engineers. With adequate performance, the relative economics of these alternate systems must be compared with the existing conventional systems, wood ties with cut spikes. In addition, the relative economic of these systems would have to be compared with several other technically proven systems, such as wood ties with alternate (elastic) fastening systems.

This economic comparison however was not a simple matter. Because different tie/fastener systems exhibited different lives, required different maintenance activities, and affected other maintenence activities differently, simple comparison of the initial or "first" cost was not adequate. Rather, a comprehensive comparison of the costs and benefits of the alternate systems, *over their entire service lives* was required. This entailed "life cycle" cost analysis of the alternate systems.

This is further complicated by the fact that component lives and behavior varied significantly as a function of track and traffic characteristics as well as individual railroad practices. Thus the relative economics of these alternate systems was not fixed, but varied with these parameters as well.

In order to develop an approach that was capable of addressing these significant differences not only between railroads, but within the railroad, it was necessary to develop a methodology for carrying out such a comprehensive life cycle cost analysis, and yet permit the variation of the key parameters that would affect the results. The approach taken to accomplish this was the development of a portable life cycle computer model that would operate on the new generation of personal computers now found in almost every railway engineering office.

---

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**Manager Analytical Engineering, ZETA-TECH Associates, Inc.

***An association that is involved in the promotion of wood crossties.

By developing an analytical methodology that was comprehensive and putting it onto a portable computer program that would allow for the changing of the key parameters and the "instantaneous" recalculation of the results, an effective means of carrying out economic benefit comparisons was implemented.

The current level of computer and software sophistication available on personal computers also allows for the development of such a program in a user friendly format, allowing the user to see all the details of this analysis and all the assumptions made in the process. This avoids a "black box" type of approach where key assumptions that drive the outcome are not readily apparent to the user. By implementing such a program in a spread sheet, the key variables are all readily apparent and easy to change, and the response to such changes, i.e. the recalculation of the results, is in real time, thus no long wait for answers. This, in turn permits the rapid evaluation of many alternatives.

This model can then be used to compare the relative economics of alternate tie fastener systems. In this case, two sets of systems were compared;

a. Wood ties with cut spike fasteners compared to concrete ties with elastic fasteners.

b. Wood Ties with elastic fasteners compared to concrete ties with elastic fasteners.

Since little new track construction has been carried out in North American in recent years and since wood ties have been the system used for well over a hundred years, the analysis was designed to assume that wood ties are in place, in track, and examines the relative economics of replacing these wood ties with the alternative concrete tie systems. This is the manner in which most concrete ties are currently installed today.

This paper will discuss the development and implementation of such a portable economic analysis model and will present some sensitivity analyses based on a specific base case example.

ANALYSIS APPROACH:

The basic analytical approach used in this methodology (and in the model itself) is a present worth analysis approach, in which all the costs associated with the two alternative systems are examined and compared in terms of a "present worth." Thus any future costs or savings associated with the two systems are brought to the present, and the "worth" of these future costs calculated using an appropriate interest rate. (Thus taking into account the time value of money.)

Therefore for each of the alternative tie/fastener systems the following category of assumed costs are compared:

1. Initial Costs Assumed.

These are costs that are incurred at year 0, i.e. the present time. These costs include such items as the initial component costs for the replacement components (the new system being installed), the cost of their installation, the cost of any accompanying work (such as the addition of ballast), and any salvage value due to components removed from the track.

2. Life Cycle Costs Assumed.

These are costs that are incurred in the future, rather than at the present. These include such costs as different maintenance costs, different operating costs (i.e. fuel), derailment risk, etc. As noted previously, these future costs must be converted into their "present" cost to allow for final summation and comparison.

These different costs are combined, in terms of today's dollars, to determine if there is a net benefit for either tie/fastener system.

Noting once again that the approach assumes that the track is originally constructed with wood cross ties and either cut spikes or elastic fasteners (depending on the comparison performed), the cost of replacing this structure with concrete ties and elastic fasteners is examined. Using the combined initial and life cycle costs, the net benefit (disadvantage) of the wood tie track is determined and presented as an output of this model. In addition, the return on investment (ROI), for the cost of conversion to

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concrete is calculated. (A negative ROI indicating that it does not pay to convert for the case examined, while a positive ROI indicating that there is an economic benefit for the conversion).

In examining life cycle component and maintenance costs, it is apparent that component lives and maintenance cycles are dependent on specific track, traffic, and operating characteristics which are specific to a given location and/or railroad. Therefore, in order to allow for such a model to be used in determining the relative economics of the different systems, it was necessary to make the model itself sensitive to these characteristics that influence lives and corresponding costs. These characteristics include:

A. Track Characteristics.

Track characteristics such as presented in Table 1 have a strong influence on the lives of the key track components such as the rail, the ties themselves, and the ballast. In addition maintenance cycles such as surfacing and grinding are determined by these characteristics. Since these differences in component lives directly affect the relative maintenance costs of the two tie/fastener systems, any effective model must include sensitivity to these track characteristics.

Table 1

Track Characteristics

- Curvature
- Grade
- Rail Section
- Wheel Load
- Lubrication
- Ballast Depth
- Tie Spacing
- Other

B. Traffic Characteristics.

In a similar manner, traffic characteristics, such as presented in Table 2, directly influence the component lives, maintenance cycles, and corresponding costs.

Table 2

Traffic Characteristics

- Speed
- Annual Tonnage
- Axle Load

C. Economic Characteristics.

Economic characteristics include such factors as the acceptable rate of interest, rate of inflation, and the railroads required rate of return for new investments, directly influence the time value of money. This, in turn, directly relates to the present worth of the future maintenance and component replacement costs.

D. Maintenance Activities.

A key area of cost differential in this analysis is that associated with life cycle maintenance costs. These costs are associated with a large group of maintenance activities such as presented in Table 3. As can be seen in that table, these maintenance activities include activities specific to either wood or concrete ties or to both. In all cases, these are differences in maintenance procedures, maintenance cycles and maintenance costs between the two systems being examined. The proper structuring of these maintenance costs and their inclusion into the comparison between the two systems represents a key part of this model and will be discussed in detail later on in this report.

Table 3

Maintenance Activities

- Rail Replacement
- Rail Transposition
- Gaging
- Tie Replacement
- Anchor Adjustment
- Surfacing
- Grinding
- Fuel Consumption
- Basic Forces

Table 4

Costs

- Initial Costs
  - Materials
  - Initial Installation
  - Conversion
  - Salvage Value
- Maintenance Costs
- Operation Costs

E. Costs.

The final major area of variability required for definition of the characteristics of the different tie/fastener systems is that of costs themselves. As presented in Table 4, these include material costs, labor costs, equipment costs, and operating costs, both directly as in the case of fastener components or indirectly such as with operating costs.

By providing for the ability to vary each of these areas, the desired level of flexibility and sensitivity is achievable in this cost benefit model.

MODEL DEVELOPMENT AND ASSUMPTIONS:

As noted previously the model(s) examines the economics of replacing an existing wood tie track system (with either cut spikes or elastic fasteners) with concrete ties and elastic fasteners. If insufficient ballast exists (or if the ballast needs undercutting) the model allows for the addition of this ballast (or undercutting of the track) at the time of conversion.

The model first examines the cost of conversion to concrete, including all material, labor, and equipment costs together with appropriate rates of production. Any salvageable materials, to include wood ties in various conditions, and conventional OTM (plates, anchors, spikes, etc.) are credited against the cost of conversion. Any disposal costs, such as for old ties that are not reusable, are added to the cost of the conversion.

Once the conversion costs are established, the individual maintenance costs are developed for each of the tie/fastener systems. These maintenance activities, which are listed in Table 3, calculate the life cycle costs associated with the tie/fastener system. Thus the model assumes that maintenance costs, such as rail relay etc., are ongoing, and that they occur at a calculated cycle. This maintenance cycle is directly dependent on the life of the component under the specified traffic and track conditions. Thus, rail relay takes place at the end of the life of the rail, which is calculated in the model. As used in this model; rail life, tie life, and surfacing cycle are calculated using equations based on the TOPS (3) equations and modified for this analysis. [Alternately, the model allows for the input of other component live values, instead of the ones automatically calculated. This provides for a user "override" for the calculated component life values.]

The maintenance activities for the two tie/fastener systems can be structured differently, using different gang composition and equipment, different materials, and exhibiting different maintenance cycles. By bringing these future maintenance costs to a present worth, they can be compared directly, and a cost difference (benefit) calculated.

Utilization of the spread sheet format allows for the variation of all values used in the calculations, thus and *all variables* can be changed.

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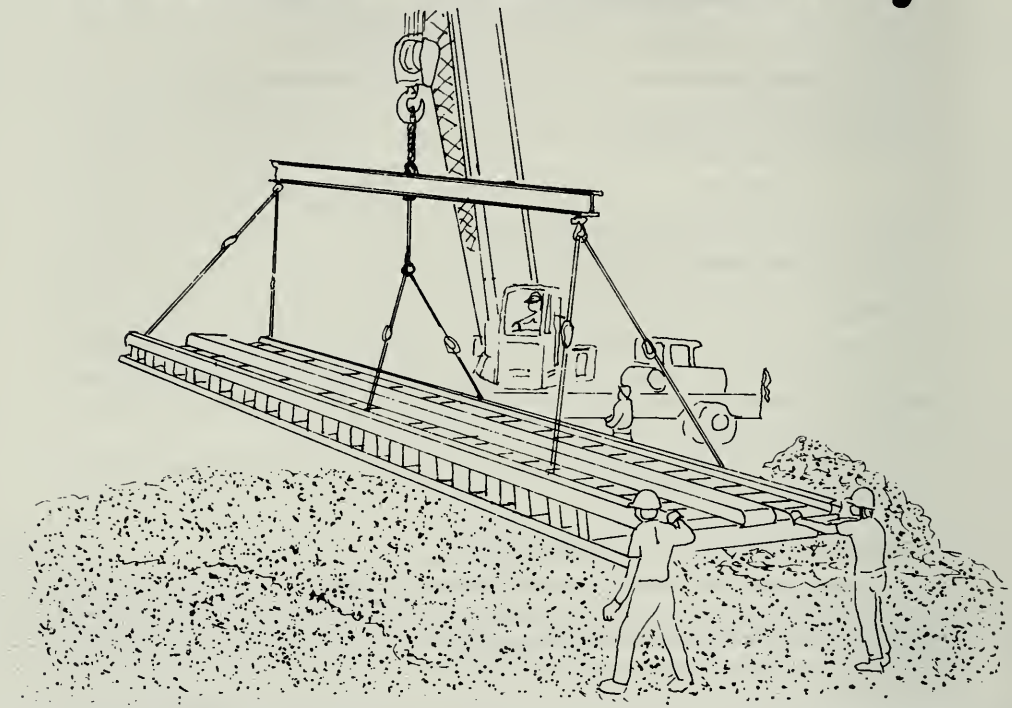

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MODEL FORMAT AND STRUCTURE:

The format of the model is designed to allow for direct comparison of the two systems on a maintenance activity by activity basis. In addition it is designed to allow for easy following of the assumptions and analysis logic.

In order to accomplish this, the model is presented in an overall format as follows:

I. Definition and Initial Information.

II. Conversion to Concrete.

III. Maintenance Costs and Comparisons.

IV. Results.

This structure allows for the ready identification and location of each of the areas of information and analysis.

The first part of the model, presents the basic information about the track structure together with other relevant cost and component information. Thus, all of the critical track characteristic information is presented in the first model screen, in the format shown in Table 5. This allows for the definition of the specific case being analyzed and the relevant track design information. In addition to this, the first section of the model presents the following additional information:

1. Calculated component lives and maintenance cycles (Table 6)
2. Economic characteristics
3. Labor rates*
4. Manpower requirements*
5. Equipment costs*
6. Maintenance productivity rates
7. Material costs (Table 7)

*assumed numbers presented in tabular form.

In addition, the overall results of the analysis are brought to this first section to allow for easy viewing of the results.

The second part of the model examines the cost of conversion of the track to concrete ties. This is a one time cost that takes place in the present time (year 0). The cost assumptions calculated in this conversion activity include:

a. Cost of equipment for conversion/installation.
b. Cost of labor.
c. Cost of new tie/fastener materials
d. Salvage value of removed materials (credit)
e. Disposal costs of removed materials (as applicable).
f. Cost of addition of ballast (if required for conversion).
g. Cost of undercutting (if required for conversion).

The third part of the model deals with the ongoing (future) maintenance costs for the two different tie/fastener systems. In this section, each of the individual activities presented in Table 3 is analyzed for the costs associated with these two systems. These costs would include labor, equipment, materials, and varying productivity rates, as applicable to the maintenance activity in question. (Note, the actual labor and equipment unit costs were presented in the first section of the model.)

In order to account for the differences in practice between wood and concrete tie systems, independent maintenance gangs are "built up" in the model, using the assumed unit costs for labor and equipment. This allows for the use of separate gangs, such as would be the case for tie replacement for wood (conventional) vs. concrete (out-of-face renewal). In addition, it allows for separate life cycles and production rates. By bringing these costs forward to a present worth, they can then be compared with each other (wood vs. concrete difference) and with other maintenance and initial costs.

Table 5

Track Characteristics Assumed

Segment Length	1 Mile
Curvature (degrees)	4
Grade	0.00%
Speed	20 MPH
Annual Tonnage	25 MGT
Rail Section	132 RE
Wheel Load	33000 LB
Type	CWR
Rail Lubricators	yes
Rail Cost	550 $/ton
Concrete Ballast Sec.	12 inch
Wood Ballast Sec.	8 inch
Number of Concrete Ties/Mile	2640
Number of Wood Ties/Mile	3200
Wood Tie Replacement Gang	700 Ties/Cycle

Table 6

Assumed Component and Maintenance Cycles

Component/Activity	Concrete	Wood
Rail Life	22.10 Years	19.91 Years
Transposing Cycle*	22.10 Years	19.91 Years
Tie Life	35.0 Years	29.03 Years
Tie Gang Cycle	N/A	6.35 Years
Rail Grinding Cycle	0.80 Years	1.20 Years
Surfacing Cycle	6.52 Years	5.65 Years
Gaging Cycle	N/A	6.35 Years
Anchor Adjustment	N/A	4.10 Years

Table 7

Assumed Material Costs

Item	Wood Ties	Concrete Ties
Elastic Fastener		$ 1.75
Tie Pad		$ 0.80
Tie	$16.74	$35.00
Insulator		$ 0.65
Cut Spike	$ 0.24	
Anchor	$ 0.72	
Plate 14″	$ 5.65	
Tie Plug	$ 0.01	

Costs Assumed for Wood Tie			Costs Assumed for Concrete Tie		
Plate 14″	2	$11.30	Elastic Fastener	4	$ 7.00
Spikes	8	$ 1.92	Tie Pad	2	$ 1.60
Anchors	2	$ 1.44	Tie	1	$35.00
Tie	1	$16.74	Insulator	4	$ 2.60
Cost/Tie		**$31.40**	**Cost/Tie**		**$46.20**
Cost/Mile	**3200**	**$100,480**	**Cost/Mile**	**2640**	**$121,968**

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Table 8

Present Worth of Cost Differences Assumed Per Mile

Cumulative Differences:	
Basic Force	($5,706)
Rail Replacement	($22,230)
Rail Transposing	($4,152)
Gaging	($6,286)
Tie Installation	($72,788)
Anchor Adjustment	($864)
Fuel	($10,732)
Surfacing	($1,128)
Rail Grinding	($1,092)
Total	($122,795)
Concrete Ties Premium Cost:	
Cost Premium Less Salvage	$95,418
Increase In Ballast Section	$4,480
Installation	$33,638
Total	**$133,536**
Net Benefit of Wood Ties based on assumptions shown	$10,741 Per Mile
Net Segment Difference in $1,000s	$11
Return on Investment for Concrete Ties Based on Assumptions Shown	**-8.04%**

The final section of the model is the results section, where the cost differences for each of the individual maintenance activities are presented together with the initial costs of conversion and installation. These items are presented as a difference between the wood and concrete tie systems on an item by item basis, as shown in Table 8 [Note, the results presented here are for a specific base case, see Table 5]. These differences are also presented graphically as shown in Figure 1.

The individual cost differences are then summed to determine the net benefit (disadvantage) of the wood tie system over the concrete tie system (see Table 8). In addition, the return on investment (ROI) of the concrete tie system (where the investment is represented by the premium conversion costs) is also presented. This permits an immediate determination of the economic viability of the concrete tie conversion option (positive ROI- yes, negative ROI -no).

Changing of any of the variables in the model results in an "immediate" (real time) recalculation of the benefits as well as permitting sensitivity and optimization studies.

SAMPLE RESULTS AND SENSITIVITIES

By design the model is capable of analyzing a very large range of cases, with the final results, the relative benefits of wood vs. concrete cross-ties *dependent on the specific assumptions and input values*. As a result it is not possible to present the entire range of possible results in this paper.

However, in order to provide an indication of the results and some of the sensitivities of the analysis, this section will present a sample set of results for a "base case" analysis. This base case is the one defined in Table 5, i.e. that of a 4 degree curve with 25 MGT of traffic. The initial comparison is that of wood tie track with cut spike fasteners compared to concrete tie track with elastic fasteners. Table 3 presents the maintenance activities used in this analysis, and the base case results, presented in Table 8 showed a net benefit to wood for this example.

Figure 1

Cumulative Savings
Concrete Tie vs. Wood Tie/Cut Spike
Based on Assumptions Indicated

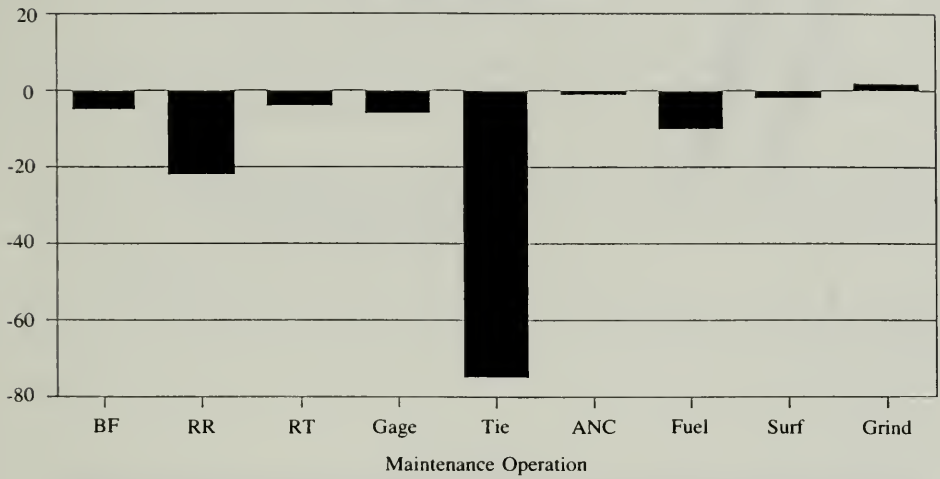
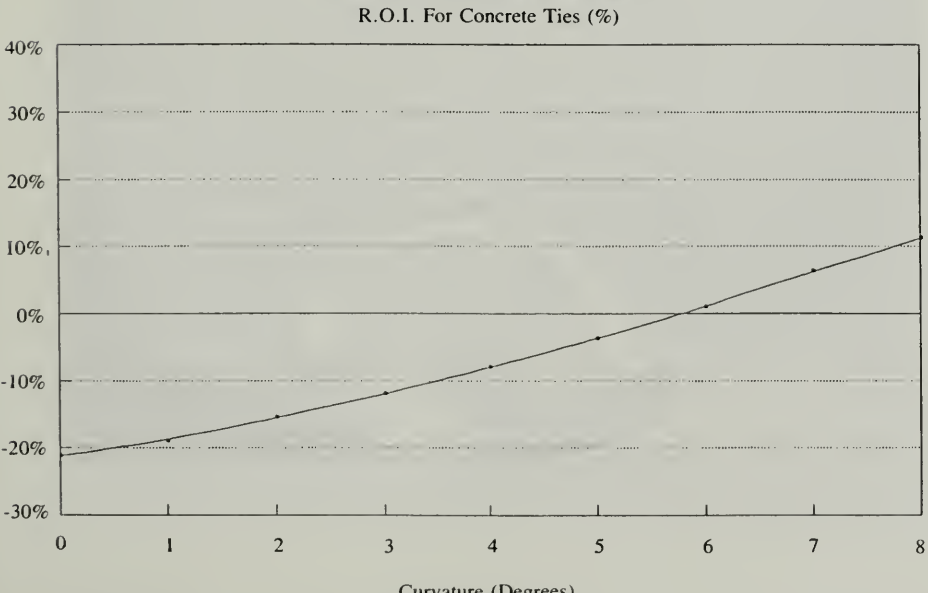


Figure 2

Economic Benefit Analysis
Wood vs Concrete Ties
Based on Assumptions Indicated



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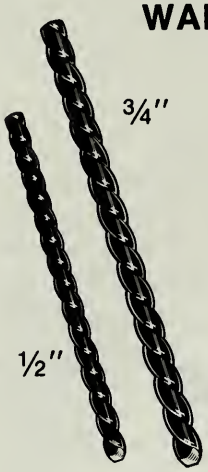

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In order to examine the behavior of the model to changes in several key variables, a set of sensitivity analyses were carried out. The results of this sensitivity analysis is presented in Figures 2 through 7. These results will be briefly discussed here. Once again, it must be noted that these results represent variations from the previously defined base case example. As variables are changed in this base case, the results of the analysis and of the sensitivity studies will also change.

Figure 2 presents the relationship between the Return on Investment (ROI) for concrete tie track as a function of the track curvature (all other variables being held constant to the base case values). As can be seen in this Figure, there is a negative ROI for concrete tie track for curvatures up until 5½ degrees. Beyond that value, the ROI for concrete becomes positive. This indicates that for this base case, wood tie track is more economic until approximately 5½ degrees after which concrete tie track becomes economically viable. This is for wood ties with cut spike fasteners.

Performing the exact same analysis for wood ties with elastic fasteners (vs. concrete ties with elastic fasteners), the results are as shown in Figure 3. In this case, the transition point, at which the ROI for concrete track goes from negative to positive is of the order of 6½ degrees. This suggests that for track where the benefits of concrete are marginal, wood ties with elastic fasteners represent an economical alternative.

The remaining sensitivity studies presented herewith address the case of wood ties with cut spike fasteners vs. concrete ties with elastic fasteners.

Figure 3

Economic Benefit Analysis
Concrete vs Wood Ties/Elastic Fasteners
Based on Assumptions Indicated

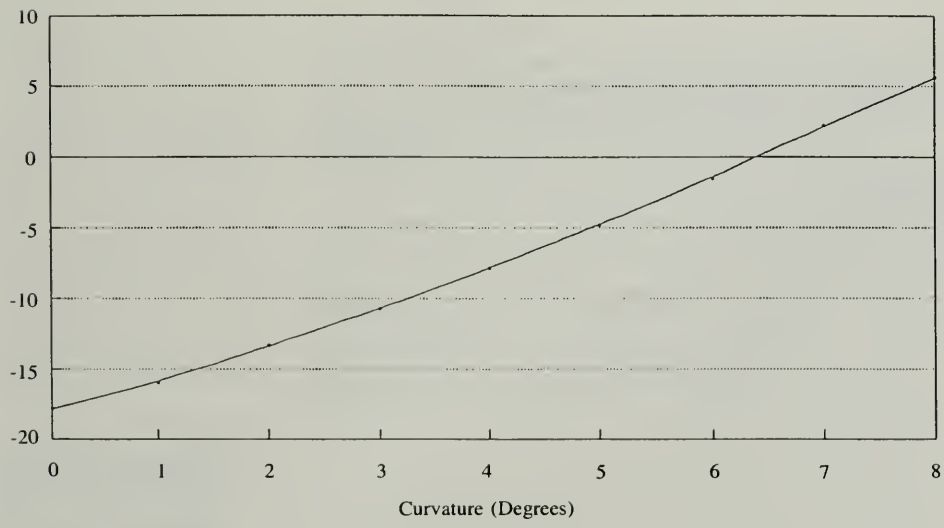


Figure 4

Economic Benefit Analysis

Wood vs Concrete Ties

Based on Assumptions Indicated

R.O.I. For Concrete Ties

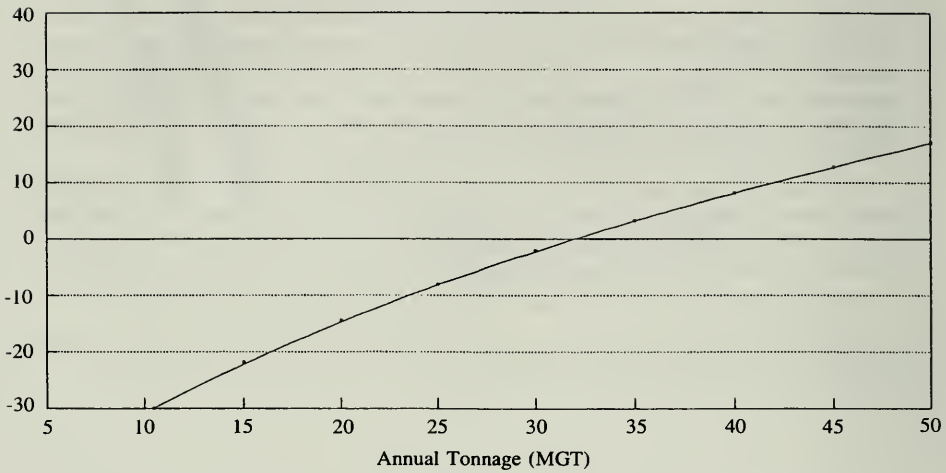


Figure 4 presents the effect of varying annual tonnage (in MGT) for the base case 4 degree curve. As can be seen in this Figure, the ROI for concrete tie track becomes positive (indicating a benefit in changing to concrete) at an annual tonnage level of approximately 32 MGT. For track with less than 32 MGT of traffic, the concrete conversion ROI is negative for the base case.

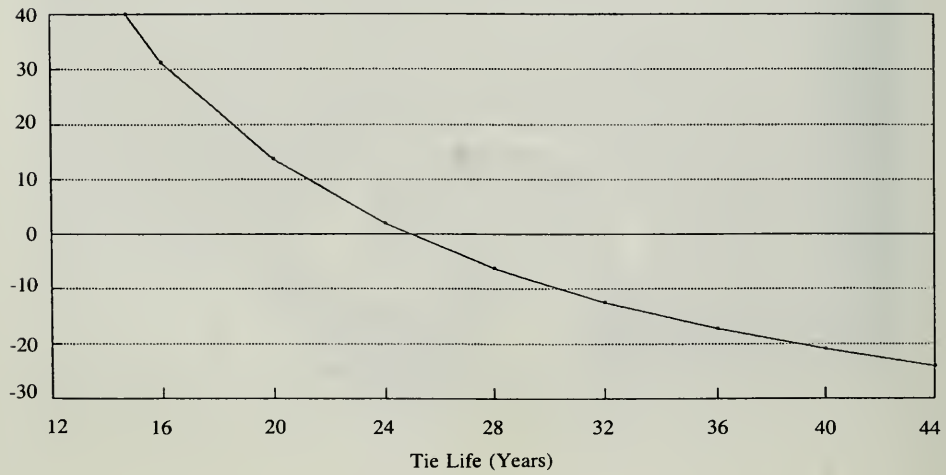
Figure 5

Economic Benefit Analysis

Wood vs Concrete Ties

Based on Assumptions Indicated

R.O.I. For Concrete Ties



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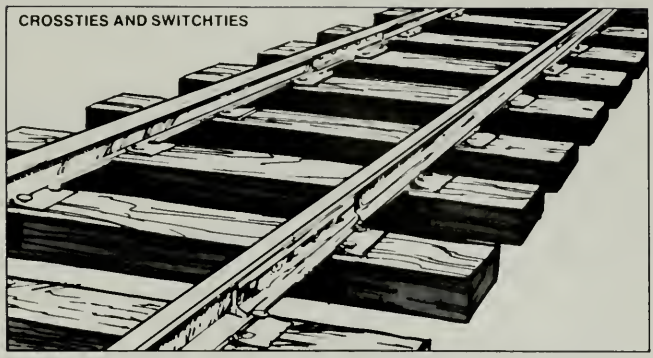

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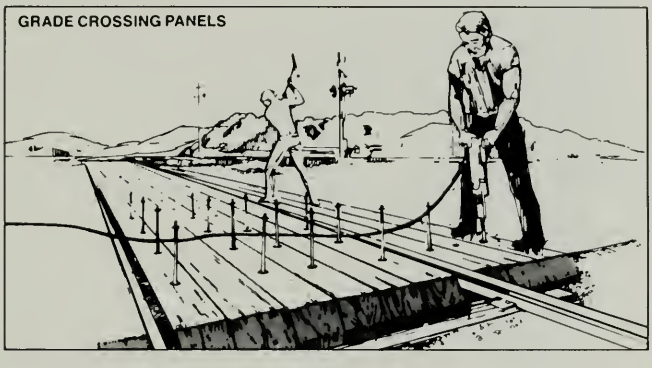

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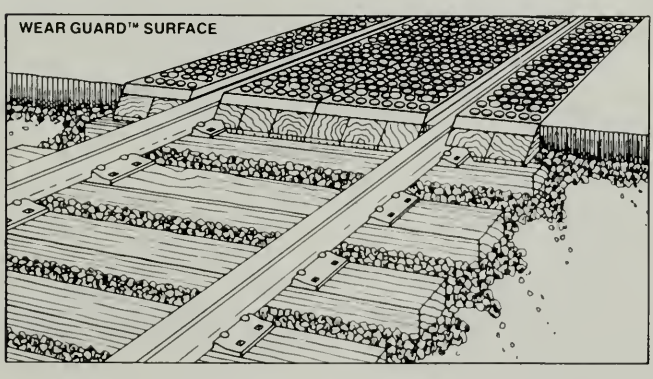

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The effect of varying wood tie life for the base case is presented in Figure 5. In this example, the ROI for concrete tie track becomes positive when wood tie life is less than 24 years. If the wood tie life is greater than 24 years, the Concrete ROI is negative.

Figure 6

Economic Benefit Analysis

Wood vs Concrete Ties
Based on Assumptions Indicated
R.O.I. For Concrete Ties

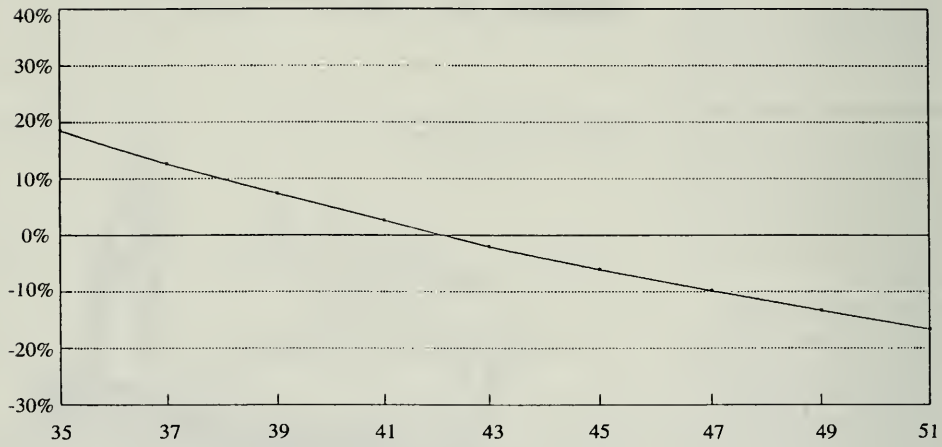
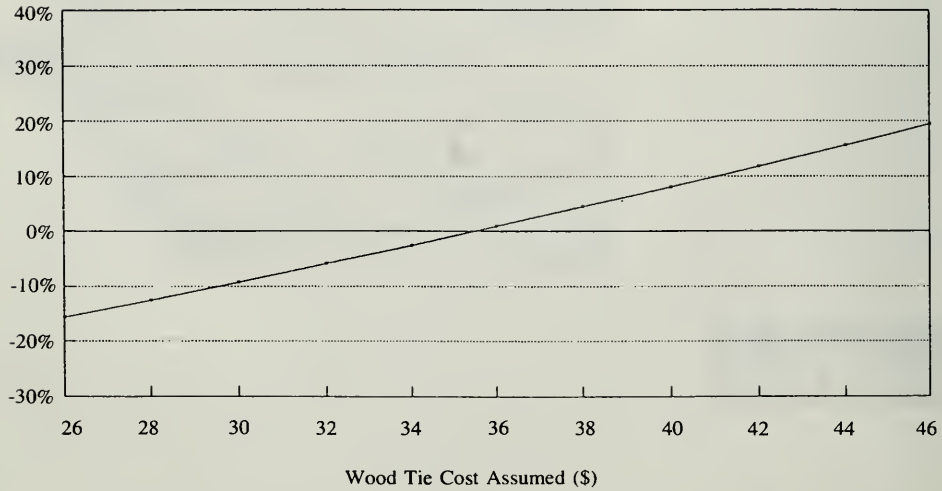


Figure 7

Economic Benefit Analysis

Wood vs Concrete Ties
Based on Assumptions Indicated
R.O.I. For Concrete Ties



Figures 6 and 7 presents the effect of varying the tie costs, for concrete and wood ties respectively. As can be seen in these Figures, the ROI for concrete tie track becomes positive as the concrete tie price decreases or when the wood tie price increases. Thus for concrete tie price levels below $42 per tie, the base case shows a positive ROI for concrete. Alternately if the wood tie price increases above $35 per tie, the ROI becomes positive for the conversion to concrete tie track.

Once again, it must be noted, that as individual variables are changed, to include track, traffic, maintenance, or cost characteristics, the specific "break even" points for the two sets of analysis will change.

CONCLUSIONS:

The results of the activity described in this paper is a user friendly computer model for the economic analysis of wood vs. concrete tie track. As such, it is not possible to simply state when concrete is economically viable or when wood is the economically attractive alternative. These decisions must be made, using the model, on a case by case basis.

However, it must be noted here, that the analyses show that there are locations and conditions under which either wood or concrete tie track is economically viable. That is, there are locations in track, where wood is the economically attractive alternative and there are locations in track where concrete is an economically attractive alternative. However, specific analysis must be carried out to define the respective boundaries.

Furthermore, it has been shown that the use of elastic fasteners on wood tie track, instead of conventional cut spike fasteners will shift that "break even" point, thus changing the conditions under which concrete track is economically viable.

In examining the applications of such an economic analysis model, the ability to perform sensitivity analyses, such as presented in the previous section, appears to have some real benefit to decision makers in both the railroad and in the supply industries. By examining the impact of changing key variables, the long term ramifications of these decisions can be examined. This also allows for the establishment of economic "boundaries" to define those locations where either track alternative is more economically attractive.

Finally by allowing for variation in track maintenance practices, to include gang size, make-up, and productivity, it is possible to examine the effects of different maintenance practices. This can be performed not only within the context of the wood vs. concrete tie studies, but individually for specific individual cases. This allows for the ultimate broadening of this type of economic analysis model to investigate the sensitivity of track costs to a wide range of variables both within and outside the control of railroad engineering departments. As such, it offers the potential for being an extremely powerful economic analysis tool.

REFERENCES:

1. Hay, W. W., *Railroad Engineering; Second Edition*, John Wiley & Sons, New York, 1982.
2. American Railway Engineering Association, *Manual for Railway Engineering*, AREA, Washington, DC, 1988.
3. Danzig, J. C. et al, "Procedures for Analyzing the Economic Costs of Railroad Roadway for Pricing Purposes," US Department of Transportation Report DOT-FR-30028, January 1976.

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The Railway Tie Association

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Use of Under Plate Treatment in Place for Ties

By: W. T. McCarthy*

BACKGROUND

In 1985 Burlington Northern approached the Osmose Company for a process to extend the life of wood ties in track without removing the tie plates or spikes. BN was finding that more than 50% of its ties on light density lines were being replaced due to decay or weathering. Both companies realized that a successful in place treating program would lead to long term economic benefits because of the extended tie lives. The biggest problem was to keep treating costs low in order to make the program cost effective.

The Osmose Company developed and patented a process to pressure inject wood preservative between a tie plate and tie through an open hole in the plate. In 1986 BN decided to implement a trial program and contracted with Osmose to provide the treating services. This involved treatment of some 151,000 ties near Springfield, Missouri and 45,000 ties near Coon Rapids, Minnesota. That same year, the Herzog Contracting Corp. entered the market with a set of it's own equipment. Their competitiveness brought treating prices down considerably resulting in a contract with BN for treatment of 63,000 ties.

The 1986 program proved that in track tie treating is effective both from an economic and engineering standpoint. Early indications showed a desire amongst contractors to perform this work at competitive prices. From the Railroads standpoint, it was felt there would be a general decline in costs as more and more contractors became involved. Laboratory tests and field investigations showed the disbursement and potency of the preservative treatment was better than originally expected. As a result BN has expanded in track tie treating programs from the original 115,000 ties to an average of 738,000 ties annually. With the increased programs, the railroad found a third treating contractor, Crisafulli J. & J..

*Director Maintenance Planning, Burlington Northern Railroad

RATIONALIZING A TIE TREATMENT PROGRAM

It is generally accepted by most Railroads that wood ties fail from decay and/or mechanical wear. A 1987 AAR Report No. WP-127[1] by the Track Maintenance Research Committee summarizes the results of tie condition surveys performed on four different railroads as follows:

> "The results of the surveys can be summarized into the following observations:
> - Decay/weathering is the major mode of tie failure in low tonnage, tangent track.
> - Mechanical mode failures increase with curvature and tonnage."

Figure 1 uses data from a statistical study of tie failure causes conducted by Tuve[2], et. al on the Southern Railway. This illustrates the relationship of annual gross tonnage to the percentage of failed ties due to either decay or mechanical deterioration.

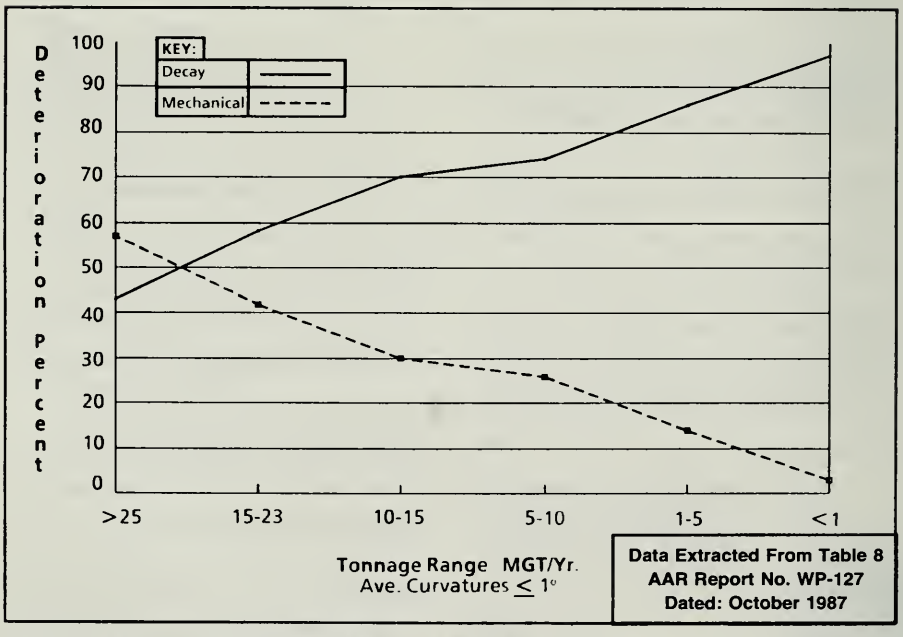

Figure 1 Results of AAR Study on Causes of Tie Failure

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- 360 degree continuous rotation
- 1,000 pound lift capacity at 24 foot radius
- Handles both wood and concrete ties with ease
- Fail-safe brake system
- 20 m.p.h. track travel speed in either direction
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Arguably, tie failure modes are determined subjectively and as a result the accuracy, consistency and repeatability of such surveys can be questioned. However; this AAR report and numerous other surveys by the AAR, AREA, the FRA and industry suppliers agree that over 40% of the ties on North American Railroads fail from decay. Additionally, most reports find that decay is more prevalent in the tie plate area because the retained moisture and exposed untreated wood promote the growth of decay fungi.

According to BN's R-1 reports, prior to 1986 it renewed an average of 4.1 million ties per year. If an average installed cost of $32.00 per tie is assumed, average annual tie renewal costs were $123 million. Using a conservative estimate that 40% or 1.64 million of the ties failed due to decay means $52.5 million was spent each year to renew decayed ties.

Table 1 illustrates the results of my present worth analysis showing the possible benefits of in track treating. Shown are the dollars saved as a result of postponing tie renewals from between 1 and 5 years. This analysis looks at three separate results assuming that treatment will be effective on either 50%, 75% or 100% of the ties which would normally fail from decay each year.

No. Ties Treated (Mill.)	Ties Treated Effectively (Mill.)	Potential Savings From Extended Tie Life (Life Extension in Years) ($ in Millions)				
		1 Year	2 Years	3 Years	4 Years	5 Years
1.64	1.64	$3.9	$7.5	$10.8	$13.9	$16.8
1.64	1.23	$2.9	$5.6	$8.1	$10.4	$12.6
1.64	0.82	$1.9	$3.7	$5.4	$7.0	$8.4

Table 1: Potential Savings From In Track Tie Treating Program

The table shows that if only one half of the ties which normally fail from decay can be maintained for an additional year, the average savings per tie is $1.90/1.64 = $1.16. If the same number of ties last an additional four years, the average savings is $5.12 per tie.

Using a similar analysis, BN found that savings from an in track tie treating program which extends tie lives at least five years would cover program costs and provide a reasonable return on investment. At this point talks began with the Osmose Company about development of a process which would effectively treat the ties at a reasonable cost.

CHOOSING A PRESERVATIVE

Based on their 15 years of successful experience treating wooden bridges, Burlington Northern asked the Osmose Company to investigate the practicality of treating ties in track. It was hoped that a fungicide either existed or could be found which would be easy to apply to the under plate area of wood ties and extend tie life by at least four years.

There has been a great deal of research into in place preservative treatment of Bridge and Marine Piling. This research was found to be applicable to effective treatments for railroad cross ties. Highley[3] found that proper treatment of the top, cut off end, of Douglas Fir Piles provides at least 5 years of protection against decay even in a high decay hazard climate. Leutritz[4] et al found that formulations having combinations of creosote, penta and sodium fluoride appeared most promising as ground line treatments for southern pine poles. Stier[5] in work done for Osmose reported that "Treating Ties with ADZ-LIFE™, should lengthen useful tie life by 4 years".

ADZ-LIFE is a chemical wood preservative containing 30% Sodium Fluoride, 15% Creosote and 55% inert ingredients. It is shipped as a dark brown granular slurry and is used without mixing with other substances. The preservative is registered with the EPA as a pesticide under No. 3008-13 because of the Sodium Fluoride and Creosote content. About 50% of the compound is water soluble and in fact the preservative needs moisture in order to disperse in wood.

The EPA requires ADZ-LIFE preservative to be labeled as a hazardous compound . In addition, many States require that applicators of ADZ-LIFE obtain certification by passing some form of written test. Some States insist on town or county approval

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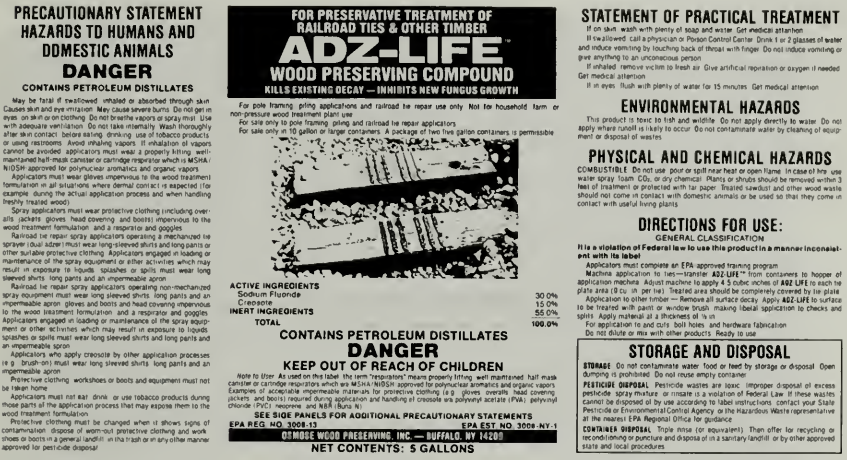

PRECAUTIONARY STATEMENT
HAZARDS TO HUMANS AND
DOMESTIC ANIMALS

DANGER

CONTAINS PETROLEUM DISTILLATES

May be fatal if swallowed, inhaled or absorbed through skin. Causes skin and eye irritation. May cause severe burns. Do not get in eyes, on skin or on clothing. Do not breathe vapors or spray mist. Use with adequate ventilation. Do not take internally. Wash thoroughly after skin contact before eating, drinking, use of tobacco products or using restrooms. Avoid inhaling vapors. If inhalation of vapors cannot be avoided, applicators must wear a properly fitting, well-maintained half-mask canister or cartridge respirator which is MSHA/NIOSH approved for polynuclear aromatics and organic vapors.

Applicators must wear gloves impervious to the wood treatment formulation in all situations where dermal contact is expected (for example, during the actual application process and when handling freshly treated wood).

Spray applicators must wear protective clothing (including overalls, jackets, gloves, head covering, and boots) impervious to the wood treatment formulation and a respirator and goggles.

Railroad tie repair spray applicators operating a mechanized tie sprayer (dual adzer) must wear long-sleeved shirts and long pants or other suitable protective clothing. Applicators engaged in loading or maintenance of the spray equipment or other activities which may result in exposure to liquids, splashes or spills must wear long sleeved shirts, long pants and an impermeable apron.

Railroad tie repair spray applicators operating non-mechanized spray equipment must wear long sleeved shirts, long pants and an impermeable apron, gloves and boots and head covering impervious to the wood treatment formulation, and a respirator and goggles. Applicators engaged in loading or maintenance of the spray equipment or other activities which may result in exposure to liquids, splashes or spills must wear long sleeved shirts and long pants and an impermeable apron.

Applicators who apply creosote by other application processes (e.g., brush-on) must wear long sleeved shirts, long pants and an impermeable apron.

Protective clothing, workshoes or boots and equipment must not be taken home.

Applicators must not eat, drink, or use tobacco products during those parts of the application process that may expose them to the wood treatment formulation.

Protective clothing must be changed when it shows signs of contamination. Dispose of worn-out protective clothing and work shoes or boots in a general landfill or in a trash or in any other manner approved for pesticide disposal.

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RAILROAD TIES & OTHER TIMBER

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WOOD PRESERVING COMPOUND

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For pole framing, piling applications and railroad tie repair use only. Not for household, farm or non-pressure wood treatment plant use.

For sale only to pole framing, piling and railroad tie repair applicators.

For sale only in 10 gallon or larger containers. A package of two five gallon containers is permissible.

ACTIVE INGREDIENTS	
Sodium Fluoride	30.0%
Creosote	15.0%
INERT INGREDIENTS	55.0%
TOTAL	100.0%

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KEEP OUT OF REACH OF CHILDREN

Note to User: As used on this label, the term "respirators" means properly fitting, well maintained, half-mask canister or cartridge respirators which are MSHA/NIOSH approved for polynuclear aromatics and organic vapors. Examples of acceptable impermeable materials for protective clothing (e.g., gloves, overalls, head covering, jackets, and boots) required during application and handling of creosote are polyvinyl acetate (PVA), polyvinyl chloride (PVC), neoprene, and NBR (Buna N).

SEE SIDE PANELS FOR ADDITIONAL PRECAUTIONARY STATEMENTS

EPA REG. NO. 3008-13 EPA EST. NO. 3008-NY-1

OSMOSE WOOD PRESERVING, INC. — BUFFALO, NY 14240

NET CONTENTS: 5 GALLONS

STATEMENT OF PRACTICAL TREATMENT

If on skin: wash with plenty of soap and water. Get medical attention.

If swallowed: call a physician or Poison Control Center. Drink 1 or 2 glasses of water and induce vomiting by touching back of throat with finger. Do not induce vomiting or give anything to an unconscious person.

If inhaled: remove victim to fresh air. Give artificial respiration or oxygen if needed. Get medical attention.

If in eyes: flush with plenty of water for 15 minutes. Get medical attention.

ENVIRONMENTAL HAZARDS

This product is toxic to fish and wildlife. Do not apply directly to water. Do not apply where runoff is likely to occur. Do not contaminate water by cleaning of equipment or disposal of wastes.

PHYSICAL AND CHEMICAL HAZARDS

COMBUSTIBLE. Do not use, pour or spill near heat or open flame. In case of fire, use water spray, foam, CO_2, or dry chemical. Plants or shrubs should be removed within 3 feet of treatment or protected with tar paper. Treated sawdust and other wood waste should not come in contact with domestic animals or be used so that they come in contact with useful living plants.

DIRECTIONS FOR USE:

GENERAL CLASSIFICATION

It is a violation of Federal law to use this product in a manner inconsistent with its label.

Applicators must complete an EPA-approved training program.

Machine application to ties—transfer ADZ-LIFE™ from containers to hopper of application machine. Adjust machine to apply 4.5 cubic inches of **ADZ-LIFE** to each tie plate area (9 cu. in. per tie). Treated area should be completely covered by tie plate.

Application to other timber — Remove all surface decay. Apply **ADZ-LIFE** to surface to be treated with paint or window brush, making liberal application to checks and splits. Apply material at a thickness of ¼ in.

For application to and cuts, bolt holes and hardware fabrication.

Do not dilute or mix with other products. Ready to use.

STORAGE AND DISPOSAL

STORAGE: Do not contaminate water, food or feed by storage or disposal. Open dumping is prohibited. Do not reuse empty container.

PESTICIDE DISPOSAL: Pesticide wastes are toxic. Improper disposal of excess pesticide, spray mixture or rinsate is a violation of Federal Law. If these wastes cannot be disposed of by use according to label instructions, contact your State Pesticide or Environmental Control Agency, or the Hazardous Waste representative at the nearest EPA Regional Office for guidance.

CONTAINER DISPOSAL: Triple rinse (or equivalent). Then offer for recycling or reconditioning, or puncture and dispose of in a sanitary landfill, or by other approved state and local procedures.

Figure 2. ADZ-LIFE Label

before ADZ-LIFE can be applied. Some, but not all of the dangers associated with use of this compound range from minor skin irritation to required medical attention depending on the degree of exposure. Similar to the dangers that have been associated with creosote.

Figure 2 is a reproduction of the label from an ADZ-LIFE container. EPA and State requirements for application, cleanup and disposal are strict and carry with them a great deal of liability if not done properly. All the same, this preservative is safer and when properly applied in the field, as found by Leutritz[4], provides more protection against the growth of decay than creosote. Of the preservatives considered, ADZ-LIFE was found to be easiest to apply and provided the highest amount of protection against the spread of decay.

EQUIPMENT AND PERSONNEL

Osmose, Herzog and Crisafulli have developed equipment to properly apply ADZ-LIFE between the tie plate and the tie. With this equipment preservative is injected in a manner that optimizes its effect on decay fungus in the plate area. Figure 3 shows preservative being pressure injected through open holes in the tie plate.

Figure 3. Photo showing preservative being injected into open plate holes. Note protective leggings.

Pressure is maintained until preservative appears at the opposite end of the plate. Random testing of recently treated ties using the Osmose system found preservative covered about 90% of the wood tie under the tie plate. This was accomplished without wasting preservative through plate holes or around the edges of the plate.

The photographs in Figure 4 illustrate a typical set of treating equipment. Personnel wearing protective clothing are injecting preservative, others are using high pressure air to clean out open plate holes. The equipment is light weight and easily moved from job to job with a 2 or 3 ton truck. Typical set up time for experienced crews is about 30 minutes. Once assembled the equipment is fast and easy to put on and off track allowing for better utilization of available track time. Again with an experienced crew, typical set on/off times are 5 minutes.

A typical in track treating gang consists of 9 men. Four men clean out tie plate holes and under the tie plate with forced air. They are followed by four men injecting preservative. One man supervises the crew, marks ties for treatment ahead of the gang and also marks treated ties behind the gang. Most gang members are licensed applicators and trade off between injection and treating in order to reduce the time they are exposed to the preservative.

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At the end of a days work, crew members clean the equipment and preservative containers in accordance with strict EPA standards. Any contaminated clothing or disposable materials are stored in used ADZ-LIFE containers for eventual movement to an EPA approved hazardous waste land fill. The contractor assumes all liability for the proper cleanup and disposal of the preservative.

Figure 4—Photo #1. Push car assembly for compressed air cleaning operation.

Figure 4—Photo #2. Push car assembly for pressurized treating operation.

GUIDELINES FOR TREATING TIES

The number of wood cross ties on Burlington Northern's mainlines is estimated to be 100 million. It would be impractical to treat all the ties on the railroad which were suffering from decay. In order to optimize the effects of a treating program, BN used the results of the research discussed above to target certain lines for treatment. Any line with less than 20 Million Gross Tons(MGT) annually and subject to extended periods of rainfall was considered a candidate for treatment. This limited the program geographically to the Pacific Northwest and South Central portions of the railroad.

Before undertaking the program a great deal of time was invested developing guidelines for identifying ties which should be treated. As anyone who has marked ties knows, it is difficult to consistently rate the condition of ties in track. Not only that, but selection of ties for treating is a complete departure from selecting ties for immediate renewal. It was determined that the only way to develop sensible guidelines was by walking miles of track. Personnel with a thorough knowledge of cross tie maintenance and preservative treatment of structural timbers worked together to develop the following simple guidelines:

A. Regardless of the amount of mechanical deterioration, ties meeting either of the following criteria will be treated:

    1. Ties judged to have at least 2 years life if left untreated.
    2. Ties showing signs of attack from decay.

B. The only exclusions to the above standards are ties falling into the following category:

    1. Ties with checks or splits in the plate area which would prevent the tie from taking treatment.

After establishing these basic guidelines, it was necessary to train the actual tie markers. The experts who developed the guidelines went with the tie markers giving them on the job training. Once trained tie markers need only periodic visits from the experts to keep up to date.

Recent outgrowths of the program have been decisions to limit treatment to trackage which has recently had a tie gang and to areas with curves less than 2

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degrees. Because it is sometimes difficult to identify a "treatable" tie, BN felt that we could more effectively handle this situation by working behind a tie gang. The decision to treat only ties in curves 2° or less was based on the amount of mechanical deterioration found in ties on curves of higher degree.

RESULTS OF FIELD TESTS

Until 1984 there were no standard procedures for taking field samples from ADZ-LIFE treated cross ties. In order to make their test results meaningful, Osmose developed such a procedure and has used it for three years. Figure 5 is an illustration of a standard tie plate showing the relation of assay samples to open and closed spike holes in the plate. The testing procedure is outlined as follows:

1. Spikes and tie plate are removed.
2. Four Sodium Fluoride(NaF) assay samples are taken near points 1 through 4 on Figure 3.
   a. Assays are made by drilling 3/8" diameter holes exactly 1/2" in depth. All drill shavings are collected using a special apparatus and combined for laboratory chemical analysis.
3. Tie plate is reset and spiked.

BN's in track tie treating program is only three years old, so the benefits are not yet known. However, test results on BN and other Railroads point favorably to the effects treatment is having on tie lives. A statistical analyses of 40 samples taken from ties on six different railroads found that Sodium Fluoride concentrations averaged 0.23 pcf with a standard deviation of 0.12. According to Fahlstrom[6], the sodium fluoride threshold value against decay fungi in creosote treated wood was determined to be about 0.08 pcf. Samples were taken from ties which were in track for at least 12 months after treatment with ADZ-LIFE.

Some of the first ties treated with ADZ-LIFE on the Burlington Northern are between Edwards, Kansas and Afton, Oklahoma. This section of track is about 90 miles long and carries about 14.4 MGT annually. There are a total of 34 curves on the line averaging 0°-20' and which comprise only 10% of the total route mileage. This is a part of the country where environmental conditions are ideal to promote decay in the ties.

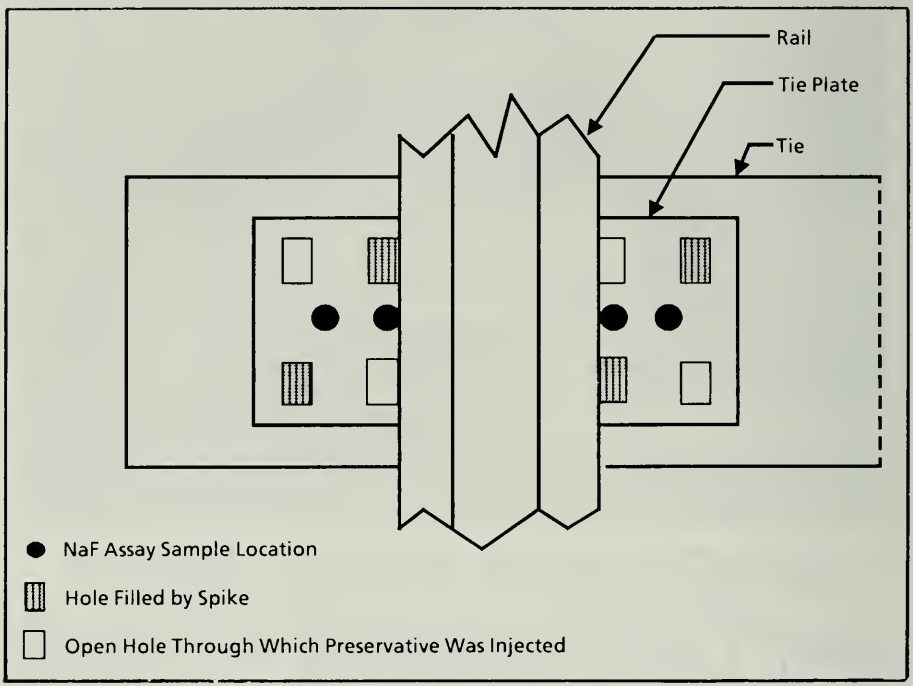
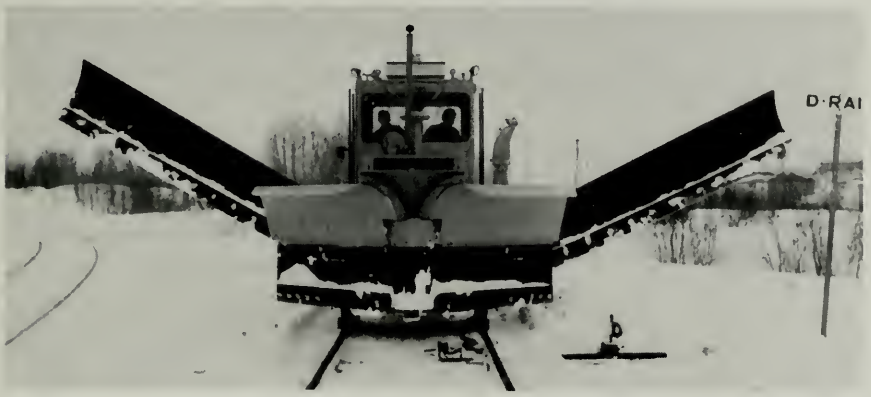


Figure 5 Illustration Showing Where NaF Samples are Removed From Under A Tie Plate

In 1986 about one third or 1000 ties per mile were scheduled to be renewed. A surfacing program using 5 cars of ballast per mile was scheduled in conjunction with the tie renewal. A decision was made to drastically reduce the number of ties to be renewed and to treat the remaining ties with ADZ-LIFE. As a result 200 ties per mile were renewed, the track was spot surfaced with about 1 car per mile of ballast, 575 ties per mile were treated. One gage spike was removed from all treated ties, the hole plugged and the spike redriven.

Reports to date from BN maintenance personnel on the Springfield Division are favorable. Since the 1986 programmed work, the track has not been surfaced and the only ties replaced have been a small number of section ties. The in track treating program in this case has allowed the reduction of planned tie renewals through the extension of wood tie lives. It is estimated the savings to date as a result of postponing the cost of tie renewal is about $670,000. This was calculated using a present worth analysis using a conservative assumption that 800 ties per mile would be renewed in 1990.

BALLAST REGULATOR & SNOW CLEARING MACHINE MODEL M



Model M Snow Clearing Machine shown with 16 foot reach hydraulically-operated snow wings, multi-position front snow plow, and snow blower, all cab-controlled.

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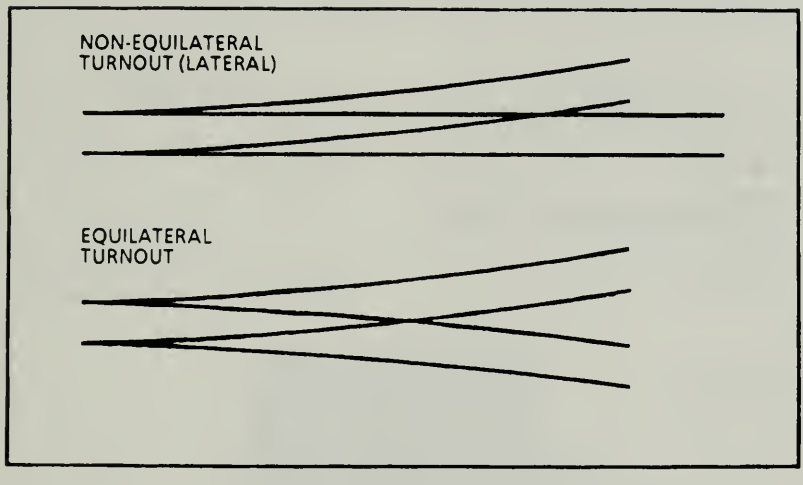
Advantages of Non-Equilateral and Equilateral Turnouts

By: S. J. Sauer*

This report is specific to heavy freight mainlines and consideration was not given for operating practices on passenger or transit operations.

First an explanation of the difference between the two types of turnouts. Non-equilateral (or lateral) and equilateral are descriptions of the geometry of a turnout. Most turnouts are non-equilateral. One side of the turnout is straight and the other is curved.

An equilateral turnout has symmetrical geometry and there is a curve on both sides of the turnout. Equilateral turnouts, when compared to lateral turnouts of the same number, allow for a higher speed on one side, and may reduce the speed on the other, but both sides are the same.



Turnouts in freight mainlines can be grouped into three categories according to their use:

1. A turnout for industry or entrance to a yard.
2. A turnout used at a passing siding.
3. A turnout used at the beginning or end of double track.

Other locations include equilateral turnouts in yards and wyes.

The three categories are in order by the speed required through the turnout. Lateral turnouts are almost always used at locations 1 and 2. At these locations, most of the train traffic will be on one side of the turnout.

A lateral turnout has the advantage of having one straight side over which trains can always operate at tangent track speeds. Lateral turnouts are always more economical for passing sidings, since the train must usually come to a complete stop in the siding.

Equilateral turnouts can not be justified at passing sidings since the majority of trains will be running on one side of the turnout and this would cause unnecessary curve wear to the rail.

*Manager Standards, Burlington Northern Railroad

Location #3 is where the decision to use a lateral or equilateral turnout must be made. The following factors must be considered when deciding between an equilateral and a lateral turnout at these locations:

A. Train speed desired through the turnout.
B. Traffic or tonnage split between the mainlines.
C. Equilateral turnouts will require maintaining inventory for another type of turnout.

This may be an easy decision in some cases, as there still may be a predominance of traffic and tonnage on one mainline.

If the traffic and tonnage are split evenly, then this may be a good location for an equilateral turnout.

At some locations, because of the unpredictability of train operations, it is difficult to determine which turnout will lead to the most economical operations.

Advantages		Disadvantages	
Lateral	**Equilateral**	**Lateral**	**Equilateral**
Obtain maximum speed thru tangent side	Maintain high speed thru both sides	Limited turnout speed	Curve wear on both sides
Easier to construct	If tonnage split is even, then this has good application	Higher wear on turnout side	More difficult to construct
Satisfactory for most situations			

Burlington Northern's current policy is to always use a lateral turnout unless there is clear justification for an equilateral.

Burlington Northern has two different sized equilateral turnouts used in our mainline #20's, and #24's.

Burlington Northern has 6-#20 equilaterals and 9-#24 equilaterals.

The maximum speed through the #20 equilateral is 45 MPH, and the #24 is rated at 50 MPH. These speeds are for both passenger and freight.

Five of the #24 equilateral turnouts are located in Southwestern Iowa, on our 50 MGT mainline between Galesburg, IL and Lincoln, NE. In 1973, BN undertook a project of converting doubletrack to alternating 10 miles of single and 10 miles of double track. The #24's were installed so that trains would not have to slow down. At that time the #24 turnouts were rated 70 MPH passenger and 60 MPH freight. In 1979, BN restricted the speed through the #24's to 50 MPH.

All of BN's equilateral turnouts use standard railbound manganese frogs.

The #20 equilateral uses a 30 foot switch alignment, similar to BN's lateral #20 turnout. No helper rods are used,but roller bearings are used. Total length of the turnout is 220 feet.

The #24 equilateral uses a 39 foot switch alignment. Helper rods and roller bearings are used on this switch. The total length of the turnout is 265 feet.

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Typical #24 frog life has been around seven years.

The following table summarizes some of the specifications of the turnouts:

Burlington Northern Equilateral Turnouts

Equilateral Turnout Number	Speed	Total Length	Degree of Curve	# of Switch Ties
20	45	220′	0°45′	143
24	50	265′	0°35′	159

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Rail Welding Policies in National Railways of Mexico

By: Ing. Gonzalo Rivera Diaz*
Ing. Carlos Davalos Marquez**

Historic Evolution of Rail Joints

The first Mexican Railroad Line inaugurated in 1873 between Mexico City and the Port of Veracruz, which had a total length of 420 km, marked the beginning of Mexico's railroad transportation. From that moment, railroad construction had a big boost, so that in 1910 there were 20,000 km. of track, from which 14,000 were of standard gage (56-1/2") and 6,000 were of narrow gage (36").

After this period railroad construction was halted due to the 1911 to 1920 armed revolution. Moreover, the revolution damaged most of the existing lines, which had to be rebuilt during the following decade. From 1940 to 1960 a few more lines were added, turning out in a small increase to the total length of the system.

Up to this point, railroad construction used joint bars to link the rails. These elements varied in type according to the rail size, the operational needs and the economic situation of that moment.

Later on, it was observed that, as a result of the increased weight in equipment, to fulfill the freight and passenger needs, several problems were caused on tracks that used joint bars. Some of these problems were faster deterioration of the ties at the rail base in the joint zone, due to the strong impact of the wheels going through. This discontinuity also caused rail misalignment, steel fatigue at the rail ends, damage to the jointing elements, and the wear out of the ballast, which during the rainy season became a major problem.

The latter problems caused deficient and unsafe train operation, damage to the equipment, excessive maintenance costs as well as uncomfortable travelling for the users.

As a result, and seeking an improvement in the operational conditions, modern railroad construction systems started to be used. These systems, which were already in use in the most advanced railways in other countries, include rail welding and new fastening devices; main elements of the elastic track.

Portion of new Chihuahua-Pacifico route.

*Budget and Technical Coordinator, Track and Telecommunications, National Railways of Mexico

**Coordinator, Office of Assistant General Director, Track and Telecommunications, National Railways of Mexico

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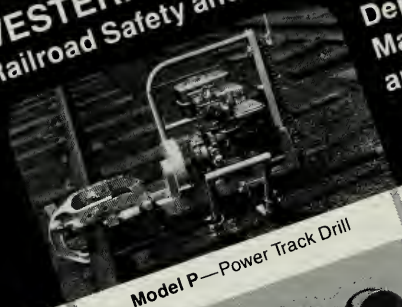
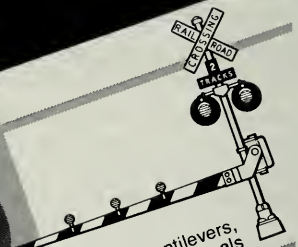


Model P—Power Track Drill



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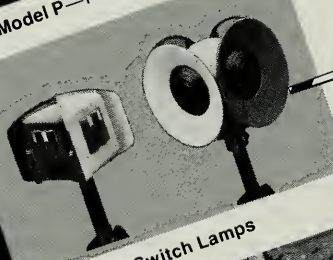
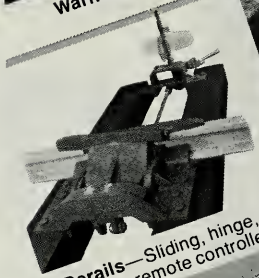
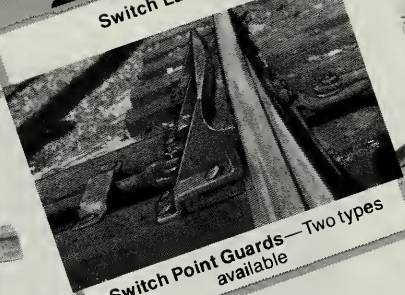


Switch Lamps



Derails—Sliding, hinge, portable, remote controlled



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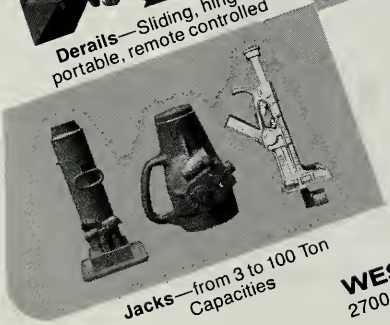

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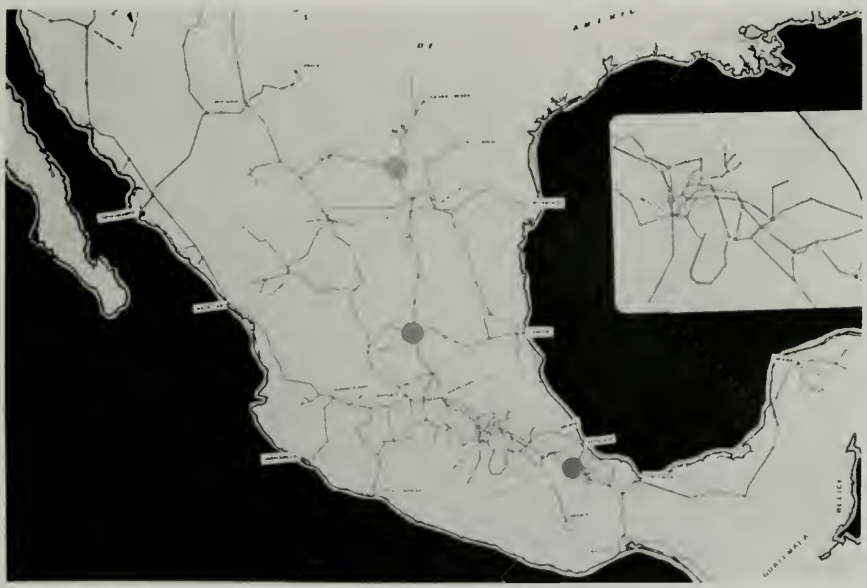
In Mexico, to renew the track segment going from Allende to Cd. Acuña, which is across the border from Del Rio, Tex., the aluminothermic technique was used in long welded rails supported by concrete ties and elastic French fastenings. Later on, the new route Chihuahua-Pacifico was inaugurated with a 166 km. segment built with the same technique.

Due to the satisfactory results obtained in the first modernizing stage, the application of this technique was intensified and used in such important lines as the Monterrey to Matamoros, and others in the Northeast and throughout the country.

It was in 1967 when the first stationary flash butt welding plant was purchased and installed at the San Luis Potosi Terminal, which was the most strategic point for the reception of standard rails acquired abroad, and the distribution of welded bars to the lines being rehabilitated.

Based on the results obtained so far, and to the increasing demand of the rehabilitation programs, it was necessary to acquire another welding plant in 1970, which was installed in Cd. Frontera, Coah. Later on, a third plant was purchased and installed in Tierra Blanca, Ver.



Locations of stationery flash butt welding plants on the Mexican National Railways.

In the period from 1980 to 1986 four mobile flash butt welding units were purchased and used for in-line jobs for the tracks in use.

In 1987 and 1988, both the San Luis and the Cd. Frontera stationary plants were subject to major repairs in order to ensure high capacity production.

Aluminothermic Welding of Rails

As it is widely known, the process of aluminothermic welding is based in an exothermic reaction of iron oxide and aluminum. When this is ignited it produces iron and aluminum oxide. The parts to be welded together are enclosed in a sand mould with a defined gap between the faces, and pre-heated to temperatures which are suitable for both the materials and the process. The parts are then welded together by allowing the molten steel produced by the reducing effect of the aluminum on the heavy metal oxide, to flow into the mould.

With iron alloys in the mixture it is possible to obtain a contributing metal with mechanical properties like the ones present on steel used for rails, obtaining a pearlitic structure.

In the first years of application of the aluminothermic welding process in Mexico, the ends of the rails to be welded were preheated at 900°C, and it was not until 1985 when a system involving light preheating was adopted. In the new system the ends were preheated between 250° and 300°C with an automatic crucible opening, using the standard 25 mm. gap, and the wide 50 mm. gap modes between the rail ends to be welded.

This welding system was used to produce the long bars with new rails of standard length of 39 ft. which would be joined later to form the continuous welded track needed for the rehabilitation programs as well as in the new products.

This system has also been used in cases where the quality of the track had to be increased by replacing the joint bars with welded joints. When it was necessary, some cropping was done in order to eliminate the damaged track segments, normally located near the bolt holes where cracks grow frequently.

Aluminothermic weld in process.

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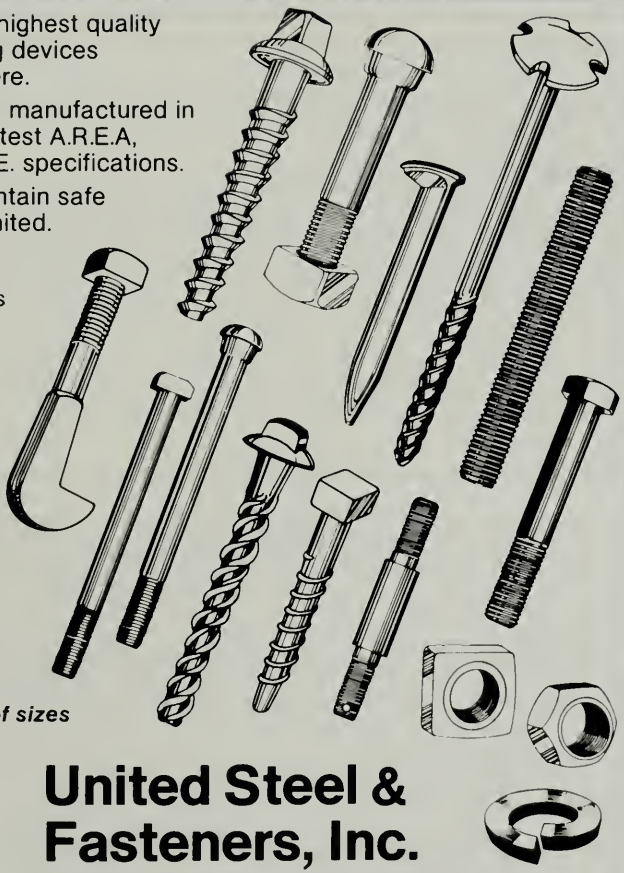

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Flash Butt Welding of Rails

This type of welding is obtained by the widely known flash butt system based on the Joule effect, using the heat generated by the resistance encountered by a low voltage, high intensity electric current while travelling through the ends of the rails to be welded. The machine moves the rails automatically to and fro against each other, so that the steel particles burn, and finally, the ends of the rails are brought to a uniform melting.

Thereafter, both ends are mechanically pressed against each other, generating the actual joining, The process consists of the following stages: square flashing, preheating, upsetting and cooling.

In Mexico since 1967 until today, this welding system has been used in the production of long rail bars for the construction of continuous welded tracks, according to the established renewal programs.

When the welding takes place at the plants, the transportation of the long bars is done on special railroad equipment, with length varying from 312 to 858 ft. depending on the geometry of the route from the plant to the unloading destination.

In the case of in-site electric welding by mobile plants, this can be done directly on the track or aside of it, using new rails or welding the used ones.

Advantages and Disadvantages of the Welding Procedures

In either case, both rail welding systems have been used with satisfactory outcome. However, each one presents different advantages and disadvantages for a specific application.

In the case of the aluminothermic welding procedure, the following advantages can be encountered:

—The welding is done specifically where required and, therefore long rail transportation is eliminated.
—The initial investment is small due to the fact that the equipment is relatively simple and easily handled.
—Fast and easy access where welding is required, making this method the most suitable for emergencies.

On the other hand, the aluminothermic technique presents the following disadvantages:

—Since this is a labor intensive process the outcome may vary depending on the ability of the different welding gangs; therefore quality control must be closely watched.
—Climate conditions may considerably affect the application results.
—When welding single railway tracks in use, the time involved in the repairs is extremely important.
—With this system it is difficult to ensure that the welding complies with the strict leveling and alignment requirements, specially in lines which have a lot of traffic.
—Another inconvenience could be the presence of slag particles and voids inside the welded section that could provoke a rail crack.
—For premium rails, the mixture dosification is very special, as well as the preheating and cooling times in order to ensure a continuity in the rail and welding hardness.

In the case of electric welding, the following advantages are found:

—The quality of the welding is uniform, due to the fact that it is an automatic machine process and manual work is minimal.
—Climate conditions do not affect the quality nor the volume of the welding production while working indoors.
—The time required for each welding is relatively small, and therefore high productivity levels are achieved in order to fulfill the established programs.
—The geometry of the welding satisfies the highest standards, because the machine can align and level the rails easily and with great accuracy.
—Low labor is required for its operation and maintenance.

Among the disadvantages observed in the electric welding process, the most important are:

—Large initial investment due to the high cost of the plants.
—Special facilities are needed in the case of stationary plants.
—Special railroad cars for the transportation of the in-plant welded rails are needed.
—In Mexico's case, where most of the system consists of single, and not signaled track lines, the cycles turn out to be very long.
—The stationary plants can only be installed where the electrical infrastructure is available.

Rail Joints in the Network. Present Conditions.

Of the 20,110 km. of main track in the Mexican Railway System, 6,900 km. are of continuous welded rail with sizes ranging from 90 to 136 lb./yd., and 2,100 km. are welded rail in small segments with the latter sizes and different lengths. The rest, 11,110 km., are of traditional jointed track with different sizes of rail. From the total of 1.52 million welded joints, 0.9 million are electrically welded, and 0.62 million are aluminothermic.

Throughout the annual inspection rounds which are part of the maintenance programs, the performance of both types of welded joints has been recorded, yielding in more aluminothermic failures than electric.

This situation has been confirmed by the readings of the rail defect detection car, which uses a double ultrasonic and magnetic system. This car has been testing our main lines since the 60's. Based on the readings, the general defect average is of 2 defects per 100 km. in the electrically welded lines, while in the aluminothermically welded lines the average is of 60 defects in the same length of track, which clearly states the reliability of electrically welded tracks for the Mexican Railway System.

Rail Welding Policies in National Railways of Mexico

Having the results of the past 28 years since joint bars were first substituted by welded joints, as a basis, it has been clearly defined where each of the two systems must be used, establishing therefore the following policies:

—Electric welding, either in plant or in-site, has priority over the aluminothermic process in all cases wherever possible.
—In all the rehabilitation cases with new rail, the long bars are formed by electric welding, either in-plant or in-site, no matter what rail length or size is involved. Since 1984 the length of the rails purchased has been 78 ft.
—The rail employed in the construction of new lines must be welded electrically, either in-plant or in-site, forming long bars.
—In the cases of track rehabilitation with relay rail, in which continuous track must be produced, all the welding involved in the production of long bars must be electric. The rails must be chosen, straightened and end cut, eliminating those segments with fatigue indications.
—In all cases where the in-plant electric welding is involved, the long bars produced must be as long as possible. The minimal length being 312 ft. and the maximum 858 ft., depending on the geometry of the route and the available train length.
—In the programs where joint bars are replaced by welded joints, cropping previously the ends, electric in-site welding is to be used.
—In the construction of new lines, or in rehabilitation of old ones, once the electrically-welded long bars have been laid, they will be welded using an aluminothermic process. In case that on the same day all the closing welds could not be done, a temporary joint bar is to be used, and must be replaced by aluminothermic weldings the next day.

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—In cases where tracks in operation are damaged by failures due to excessive thermal stresses, or in those where the rail presents a crack caused by extraordinary loads or accidents, the repairs will be done with aluminothermic welding.

—In special locations like turnouts, railway crossings, and in general all sites where electric welding is not possible, aluminothermic welding must be used.

Basic Physical and Geometric Specifications

As much as geometric tolerances are concerned, the specifications of the National Railways are akin to those of Chapter 4, Section 2 of the A.R.E.A. Manual, accepting a maximum vertical misalignment of 1.5 mm. in a one meter arc and on the horizontal plane a maximum of 1 mm. in the same length.

In order to achieve the adequate positioning molding, preheating and finishing of the rails, the National Railways Manual of Procedures is based on their own experience and on the suppliers' recommendations.

In relation to rail hardness in the welded section, it is specified that it must be 15 to 20 HBN above the hardness of the rail to be welded in the case of carbon steel rails, and in the case of high strength rails, the hardness must be the same or 10 HBN higher than the rail hardness.

For quality control purposes, 2 samples per lots of 500 welds are chosen at random in order to check Brinnell hardness tests. Ultimate strength bending test, macrographic analysis, and ultrasonic manual inspection.

Quality control for the electrically-welded rails includes the same tests mentioned above.

In the stationary plant for electric welding in Tierra Blanca, Ver., magnetic particle inspection test is also available.

Conclusions

According to the ambitious track rehabilitation programs that are now being contemplated for medium and long terms, rail welding is one of the most important objectives. It is considered that in the following 6 years a total of 7,200 km. may be rehabilitated with new welded rail, besides the welding required in secondary lines.

Currently we count with the electric welding plants required to fulfill this project, and aluminothermic welding will only be used for those special cases where electrical is not possible.

The aluminothermic welding system is actually a steel pouring operation performed in the field and whose quality is highly dependent on the human skills. For this reason, the National Railways of Mexico has paid special attention to the technical training of its' personnel who is in charge of such works, in order to ensure the maximum quality of rail welding.

Connecticut Moveable Railroad Bridge Rehabilitation Program

By: David W. Jacobs, P.E.*

A few years ago, Metro-North Commuter Railroad Company, with financial assistance primarily from the Connecticut Department of Transportation (and supplemented funding from the Urban Mass Transportation Administration and the Federal Railroad Administration) embarked on the largest moveable bridge rehabilitation project ever undertaken by an operating railroad. What makes this program unique is not the type of repairs — there's nothing particularly "state-of-the-art" or "never done before" type of work involved. What does make the project unique, however, is the scope, or extent, of work that is being performed simultaneously on four bridges, simultaneously also with other Metro-North capital and maintenance projects, and all the while maintaining:

- Better than 90% on-time performance during both peak and off-peak service on the second largest commuter railroad in the United States, and
- A minimum disruption to important commercial and recreational marine traffic on all the rivers crossed by the bridges. (Marine traffic will be affected during periods when due to electrical and mechanical work on the moveable spans, the spans will be in the down position and inoperable.)

Before I go into details of the project itself, let me give you a little background about Metro-North and how it relates to the moveable bridge project. Metro-North is the nation's second largest commuter railroad, with over 185,000 daily riders, 665 miles of track and more than 700 passenger cars. Created in 1983, Metro-North traces its roots back to the New York and Harlem Railroad, begun in 1832 as a horse-car line in lower Manhattan. The present New Haven line traces its heritage to the New York and New Haven Railroad, which opened in the 1840's between these two cities. Today, Metro-North's three major lines — the Hudson, Harlem and New Haven, operate into and out of Grand Central Terminal, in the heart of Manhattan, and fan out over six counties in New York State and Connecticut.

The New Haven line follows the same route as the Harlem line north to the New York City line in the Bronx, where it diverges and runs northeast 60 miles to New Haven, Connecticut, skirting Long Island Sound most of the way. The line runs through Westchester County in New York State and Fairfield and New Haven Counties in Connecticut, serving a total of 64 main line and branch line stations. Unlike the Harlem and Hudson lines, the New Haven line has local branch lines — to New Canaan, Danbury and Waterbury, Connecticut. An overhead 60 Hz, 13.5 KV catenary system delivers motive power to the line and also distinguishes the New Haven main line and its New Canaan branch line from the railroad's two other principal lines.

Formed by the geologic process of deglaciation, southern Connecticut has what is known as a drowned indented coastline. This is characterized by continuous irregularities: bays, inlets, points, rivers, estuaries, etc. Commercially this was of great value to the development of the state in general and the shoreline areas in particular. It provided natural breeding grounds and habitats for an almost infinite variety of marine species. More importantly, it provided ideal facilities for ports, terminals and other marine transportation systems.

As the industrial economy of the United States matured during the nineteenth century, railroads became of increasingly greater importance. Unfortunately, the very characteristics which made it possible for Connecticut to practically lead the young country into the industrial age, presented a formidable challenge to railroad builders. Unlike many inland and southern areas of the nation, the railroads of southern Connecticut, if they were to be built, had to continuously cross difficult waterways. This made railroad construction slow, expensive and, where the waterways were particularly wide and/or deep, technologically difficult.

Between the state borders with its two neighbors, New York on the west and Rhode Island on the east, approximately 130 miles, there are ten rivers which were wide and deep enough so that by the

*Assistant Director-Structural Engineering, Metro-North Commuter Railroad

early nineteenth century they were already significant commercial marine highways which could not be obstructed by artificial barriers. The first rail route along the shoreline simply provided ferries to take rail cars across the rivers. This, obviously, was slow, and during winter when the waterways often became clogged with ice, unreliable. Eventually, bridge engineers constructed moveable bridges at some of the rivers. While this was an improvement at the crossings where they were built, several of the rivers were simply too deep and the currents too swift for existing construction technology to deal with.

As the area's population and industrial base expanded during the latter half of the nineteenth century, pressure of the railroad systems to modernize and expand also grew. The shoreline route, originally made up of several independent railroad companies, was consolidated into the New York, New Haven and Hartford Railroad Company. Physically, the line expanded from a single track to four tracks between New York and New Haven and two from New Haven east. By the 1880s the last of the ten major crossings, the Thames River between New London and Groton, was finally bridged. A continuous rail link finally existed across all of southern Connecticut.

Of the ten major waterways, five are located between the New York border and New Haven — the line now owned by the state of Connecticut and operated by Metro-North Commuter Railroad Company. These rivers (from west to east) and their railroad bridge names are:

Mianus River	(Greenwich)	COB
Norwalk River	(Norwalk)	WALK
Saugatuck River	(Westport)	SAGA
Pequomack River	(Bridgeport)	PECK
Housatonic River	(Stratford-Milford)	DEVON

All five structures carry Metro-north's four-track electrified mainline. Further, all five were constructed within a ten year period — 1898 to 1908. These bridges carry over 200 trains daily; Metro-North, Amtrak and Conrail freight. Between Metro-North and Amtrak, approximately 75,000 people per day cross these bridges. Structurally the bridges are as follows:

COB
- 2 double track rolling lift bascule spans, open deck girders
- 12 open deck girder approach spans
- 28 deck truss approach spans
- Total length of structure: 1,087 feet

WALK
- 1 four-track center pivot horizontal swing span, deck truss
- 12 open deck truss approach spans
- Total length of structure: 564 feet

SAGA
- 2 double track rolling lift bascule spans, open deck girders
- 20 open deck girder approach spans
- Total length of structure: 458 feet

PECK
- 2 double track rolling life bascule spans, open thru girders
- 4 open deck girder approach spans
- 4 double track open thru girder approach spans
- Total length of structure: 366 feet

DEVON
- 2 double track rolling lift bascule spans, three trusses
- 4 open deck girder approach spans
- 8 open thru truss approach spans
- Total length of structure: 1,067 feet

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What we have then is a total of 9 moveable spans and 92 fixed spans, all between 80 and 90 years old, carrying four mainline tracks, a total of 2,475 track feet, or approximately 9,000 bridge feet — almost 1.9 miles. One can now readily appreciate the magnitude of the problem of developing a rehabilitation program, encompassing all five bridges simultaneously, around an operating commuter railroad.

COB drawbridge, Greenwich Connecticut (Mianus River)

Because of both budget and railroad operating constraints, the rehabilitation of four bridges had to be scheduled in two distinct phases. Phase I included two items: (a) the repairs to structural, mechanical and electrical systems of the moveable spans, and (b) those items not on the moveable spans but which were deemed particularly critical due to their existing condition. Phase I is under construction now and Phase II, which is the rehabilitation of the remainder of the work not included under Phase I, is scheduled to begin later this year. Phase II includes principally structural work on the approach spans, piers, fender systems and painting.

The condition of the fifth bridge, PECK (at Bridgeport), was evaluated to be no longer cost effective to rehabilitate. Therefore, a completely new structure will be built. This decision was made primarily due to a serious foundation problem with the large pit pier. Preliminary engineering is just beginning and a decision has not yet been made as to whether the new bridge will be a moveable or fixed span, or whether the alignment will be shifted or kept essentially as is.

Probably the most difficult part of this project was scheduling. In spite of the high priority given to the project by Metro-North and the Connecticut Department of Transportation (CDOT), track outages and navigation restrictions (periods when the spans cannot be opened) had to be minimized and coordinated with other capital projects involving track, catenary, signal and other bridges. Further, there was routine maintenance and non-railroad projects affecting Metro-North which also required track outages. Finally, and just as importantly, both Metro-North and the CDOT were committed to maintaining a better than ninety percent on-time performance during peak and off-peak periods.

DEVON drawbridge, Milford Connecticut (Housatonic River)

Based on all these legitimate requirements, a detailed schedule finally evolved which calls for all work under Phase I to be done within a two year period. Track outage and navigation restriction work was scheduled first at COB and DEVON. This was because these bridges were at opposite ends of the Metro-North system,and thus resulted in a minimized accumulation of possible train delays. SAGA and WALK will be done second, and simultaneously. This was done because WALK and SAGA are within the same signal block. Thus, the operating problems during track outage periods are simplified.

Another scheduling problem we had to work around was the fact that three of the four bridges, that is all except for WALK, have two identical side by side two track moveable spans. When one span is take out-of-service, two tracks are affected. The exception to this is WALK where, because it is a horizontal swing bridge, when the span is in the open position all four tracks are affected. Therefore, track outages had to be very carefully scheduled so that when two tracks were out-of-service it was coordinated with other railroad projects which needed the same tracks to be out-of-service for their work. Before work could begin, Metro-North required the contractor to certify that all materials were on-hand and ready to be installed. Further, all equipment that was to be required had to be checked out and on the site before the track outages were granted.

The problem of track outages on Metro-North is particularly critical because of its high-level passenger platforms. When one of the tracks adjacent to a platform is taken out-of-service, ramps must be installed to allow passengers to walk over the dead track from the platform to the cars. Sometimes during 2-track outages, these ramps must be installed over two adjacent tracks. Sufficient ramps must be provided so that train dwell times are not excessive due to delays caused by loading and/or unloading passengers using the ramps. Because the entire New Haven line has a TCS signal system on all tracks, we are able to further reduce delays during peak-hours, when two tracks are out-of-service, by using the in-service track adjacent to the high level platform even when its direction is opposite the normal direction. Obviously, this bridge rehabilitation project required extensive operational planning and

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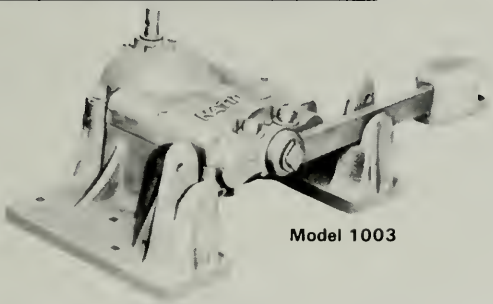

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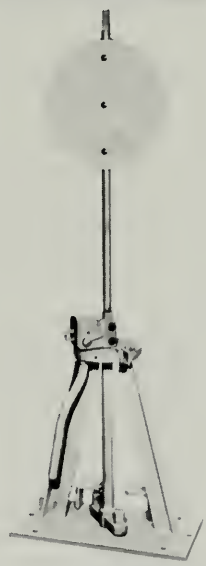

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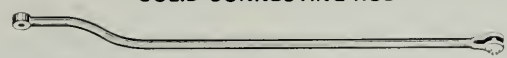

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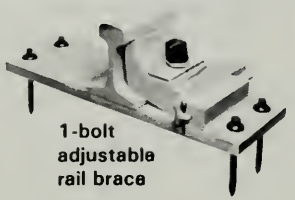

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coordination with other departments within the railroad. The same type of planning and coordination will be required for the Phase II work, since that too will require extensive track outages.

In Phase I, the primary work to be performed at the three bascule bridges (COB, SAGA and DEVON) consists of:

- Repairs to stringers and floor beams on the moveable span.
- Repairs to gusset plates, pins, rivets and other connection details.
- Replacement of flange angles and repairs to webs of track girders and segmental girders.
- Replacement of track girder and segmental girder tread plates.
- Replacement of all electrical operating motors, controllers and wiring (including navigation lighting).
- Repairs to heel and toe locks and associated mechanical/electrical systems.
- Rehabilitation of main span drives and associated mechanical systems.

The work at WALK is similar except that, since it is a horizontal swing rather than a rolling bascule, the circular tread plate track and bearing wheels (since there is no segmental girder) are to be replaced.

Under Phase II, the primary work on all four bridges consist of:

- Structural repairs to all fixed approach spans.
- Masonry repairs to all piers and abutments.
- Replacement of wooden fender systems.
- Repairs to the operator's houses.
- Installation of new inspection walkways.
- Complete sand blasting and painting.

As of this date, work on Phase I is approximately 40% complete. Work on Phase II is scheduled to begin during the summer of 1988, so that there will be some overlap between the two projects, although not at the same bridges. The total cost of Phases I and II is approximately $45 million, including design, construction, construction management and force account charges.

The final phase of the Metro-North moveable bridge rehabilitation program is, as mentioned earlier, the replacement of PECK drawbridge in Bridgeport. Based on several years of investigation, it was determined that it would not be cost effective to rehabilitate PECK and achieve a service life consistent with the other four moveable bridges after their rehabilitation. Therefore, last year the Connecticut Department of Transportation approved the consultant's recommendation regarding replacement rather than repair, and authorized Metro-North to begin feasibility studies and other preliminary engineering activities.

Frankly, it is Metro-North's desire to replace the existing drawbridge with a new fixed bridge. The savings in design, construction, operations and maintenance (in both the short and long terms) are obvious. Preliminary designs for both fixed and moveable spans are being prepared and studied, and negotiations are being held with the Coast Guard and other federal, state, local and private interests regarding proposed horizontal and vertical clearances, location of piers and possible change in right-of-way alignment. This preliminary phase of the PECK project is scheduled to be completed in April of 1988, resulting in a final selection of bridge type, geometry and alignment. Following the required regulatory reviews and approvals, we will begin final detailed design. It is hoped that construction can begin in 1991.

Foundation Stabilization of a Bridge Pier Using Passive Shear Pins

By: N. I. Norrish*, A. J. Krause* and J. E. Swanson**

Introduction:

Bridge 355.0 is located on the Alaska Railroad Corporation (ARRC) mainline approximately five miles south of Healy, Alaska (Figure 1). The bridge is a vital link for rail traffic between Fairbanks and Anchorage and also for unit coal trains which originate at the Usibelli Mine and cross the bridge en route to the port of Seward.

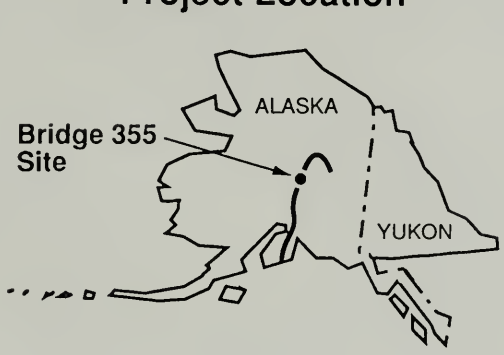

Figure 1

Constructed in 1949, the bridge is a two-span structure consisting of a 123 foot pony truss and a 55-foot deck girder. The center pier is a 50-foot tall, cast-in-place concrete pier keyed into existing bedrock. (Figure 2).

Bridge History

The initial crossing of Antler Creek at Mile 355.0 by the ARRC mainline was accomplished by a trestle bridge constructed in 1921. The foundation for this bridge was composed of low, stepped concrete piers on fourteen-foot centers, with a central clear span of 28 feet which crossed the creek. A timber trestle was built up from these piers. Remnant portions of these earlier piers can be seen today in various stages of deterioration. In some cases the trestle piers are in the process of being overturned by lateral earth pressures.

According to ARRC records, a World War II Army surplus standard truss bridge section was selected to replace the wooden trestle shortly after the war. Because the width of the canyon exceeded the length of the bridge span, a center pier was constructed on the bedrock exposed along the northern slope of the creek. Approximately 75 percent of the base of the pier was constructed to be in contact with the in situ rock on a level surface while the remaining 25 percent of the base was constructed parallel to the slope. Concrete abutments were poured on each end of the superstructure to complete the bridge unit.

According to ARRC survey records, the pier has experienced minor horizontal and vertical movement over the years and has presented periodic maintenance problems to bridge and building (B&B) maintenance crews. These movements necessitated shimming at the center pier to compensate

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Figure 2. General view of bridge looking toward the South.

for the vertical movement (Figure 3) and notching of the south concrete headwall to accommodate horizontal movement and to relieve the resulting compressive stresses in the truss members. Since this movement appeared to be a gradual but continuous creep, the symptoms were accommodated without great concern. During and subsequent to a heavy rainfall in October 1986, the ARRC District

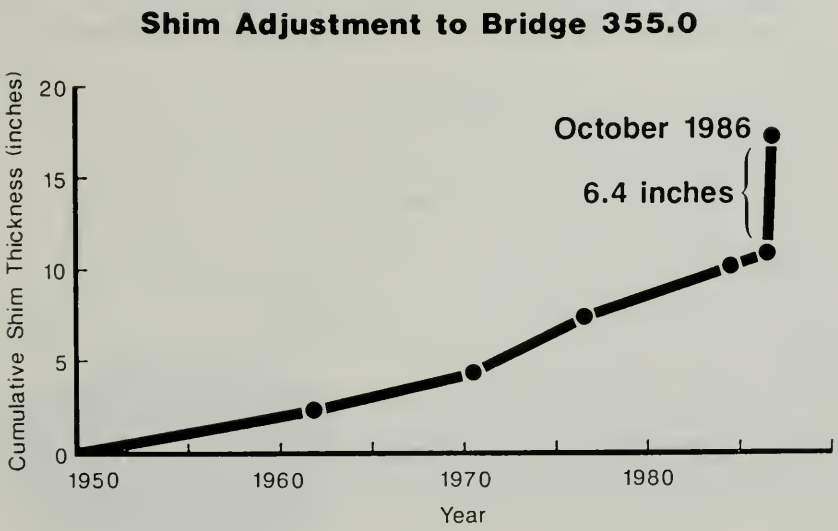


Figure 3

Roadmaster noted major shifting of the bridge. A damage survey team consisting of ARRC and Federal Emergency Management Agency (FEMA) personnel examined the movement and confirmed the Roadmaster's observations. Due to the severity of the movements, the potential for catastrophic failure, and the importance of the structure to mainline traffic, ARRC commissioned an engineering evaluation to analyze the cause of failure and to design suitable remedial measures.

During and subsequent to the October 1986 flood, the center pier moved vertically (greater than 6 inches) over a short period of time. Incremental horizontal movement of the pier as a result of this storm event is not known because the lack of clearance which existed between the bridge structure and south abutment prevented movement of the top of the pier. Limited survey information collected after the October 1986 flood indicated the pier accommodated base movement by leaning the top 0.48 feet north of vertical. Additionally, various Bridge 355.0 measurements were taken by Electronic Distance Measurement (EDM) for comparison to as-built dimensions (Figure 4). These measurements indicated the pier has experienced about 8 inches of horizontal movement since its construction in 1949 compared with the 17 inches of vertical movement indicated by the shimming records (Figure 3).

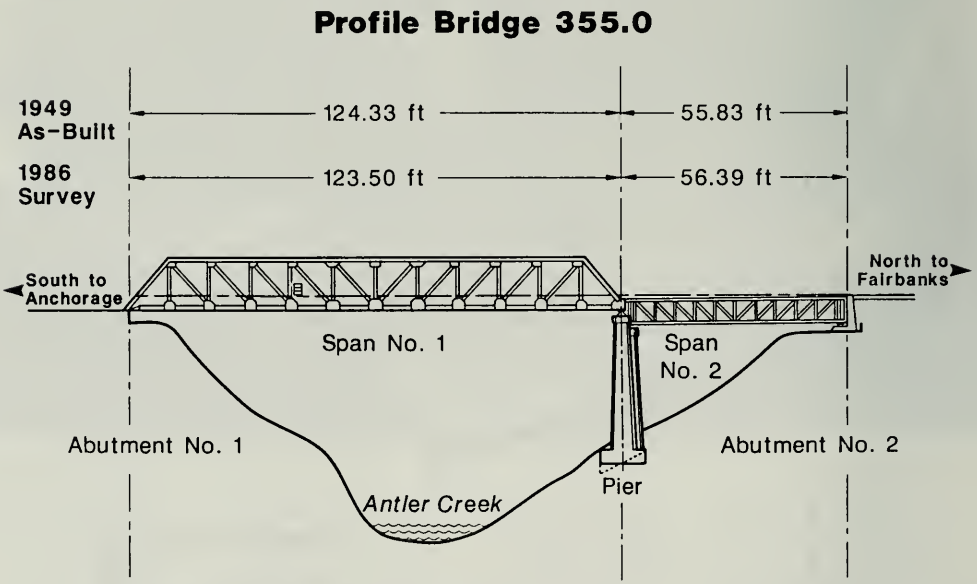
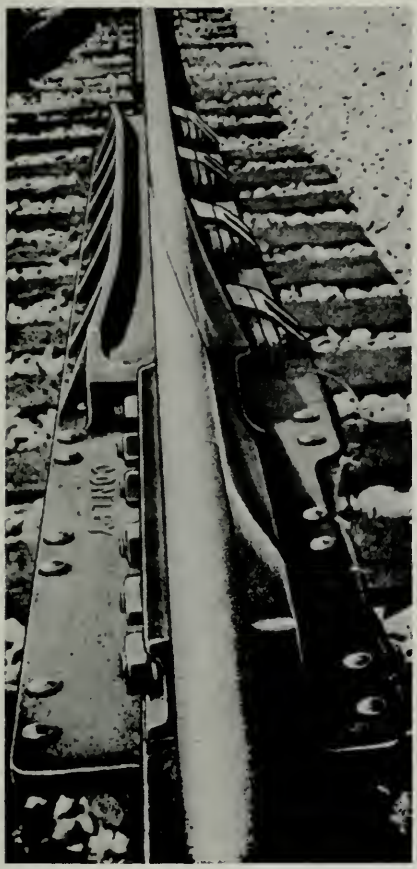


Figure 4

Site Conditions

Bridge 355.0 is situated in a northeast-southwest trending, steep-sided gulley cut by Antler Creek into the west valley wall of the Nenana River (Figure 5). The north bank of Antler Creek, on which the central pier is located, has an overall average inclination of about 38 degrees and represents a dip slope for the foliation planes within the rock mass.

The pre-Cambrian Birch Creek Schist formation is the foundation material underlying Bridge 355.0. This formation is an east trending metamorphic zone approximately 12 miles wide through the Healy Canyon. The area is characterized by active tectonism including metamorphic alteration of the in-situ rock and faulting associated with the east-west trending Denali fault system.



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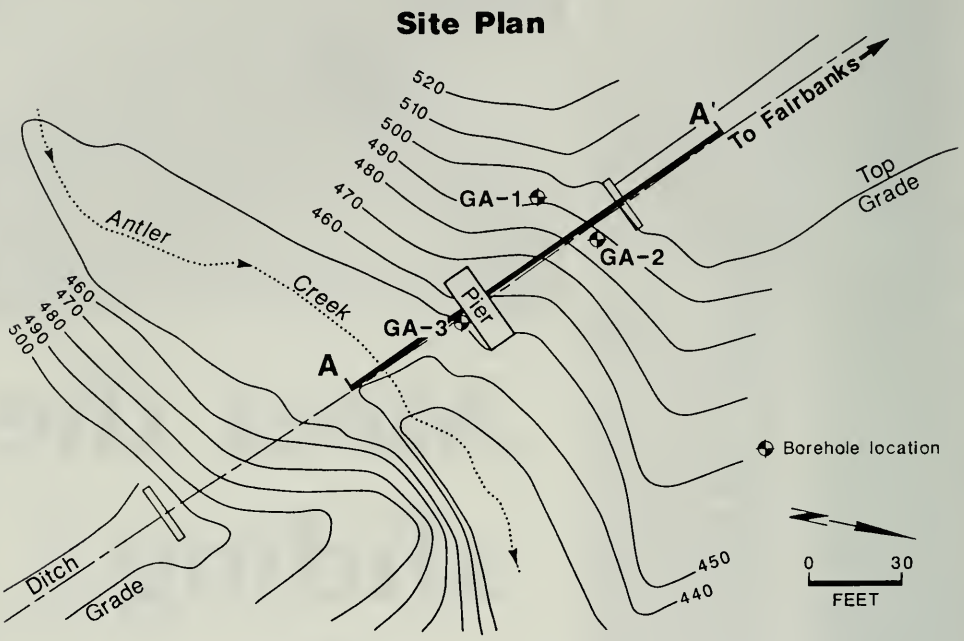
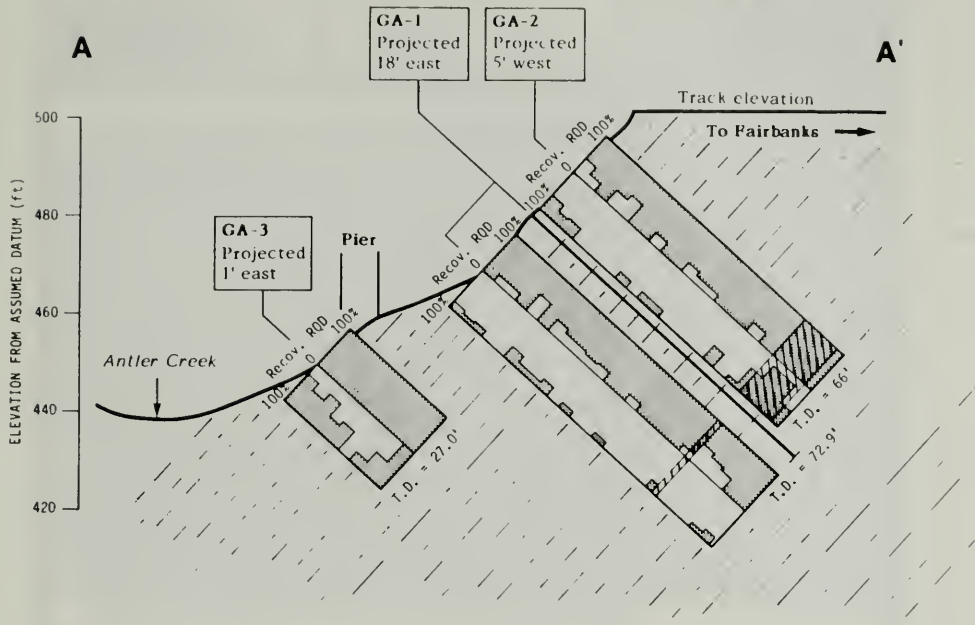


Figure 5

In order to investigate subsurface conditions, three exploration holes were advanced with a portable Mobile drill. Figure 6 illustrates the variation in rock quality with depth based on evaluations of the Rock Quality Designation (RQD) index and the core recovery index in the three boreholes. Based on these indices, the engineering quality of the rock mass generally rated as poor to fair with local zones classified as very poor. Based on the core drilling, the overall rock quality appeared to improve below the highly weathered surficial zone which is about 10 feet thick. Soft core intervals, which may represent shear zones, were noted in two of the boreholes (see Figure 6), while the third hole was typified by very poor rock conditions which eventually caused the hole to be prematurely terminated.

Weathering intensity ranged from severe at the surface to generally moderate at depth. Local zones of severe weathering were also encountered at depth within the rock mass. Exposed Birch Creek Schist at the project area has weathered to a brown or reddish brown color, resulting in a reddish stained schistose rock (Figure 7). Weathering of the mica facilities spalling along plans of foliation which leads to the formation of slabs and flakes bounded by cross joints. Frost action, enhanced by chemical weathering, has broken up the schist into blocks of various sizes that have accumulated rapidly as talus at the base of the slope.

Discontinuity orientation data, including foliation, joints and faults which were mapped at Bridge 355.0, were consistent with the regional structural geology. Foliation is well developed in the schist, and although not uniform, generally strikes eastward and dips 26 to 45 degrees southward; it is locally very irregular and highly contorted. Joint mapping data indicated a broad range of orientations with a dominant orientation for the well developed cross-joints of 65 degree to vertical dip toward the west. Figure 8 illustrates an isometric view of the idealized structural geology at the bridge site.



NOTES:

1. RQD index to engineering quality of rock

25%	Very poor
25-50%	Poor
50-75%	Fair
75-90%	Good
90-100%	Very Good

2. RQD and Core Recovery histograms are reverse shaded so that poorer quality rock is indicated by dark shading.
3. / / / - soft core intervals which may represent shear zones
4. For section location see Figure 5.

Figure 6
Section A - A'

Failure Mechanisms

Based on the results of the structural mapping and the drilling program, potential mechanisms of failure were developed for the Bridge 355.0 pier slope. The following salient geotechnical factors were used to derive failure models for engineering analyses:

- The Birch Creek Schist is inherently weak because of separation along foliation planes, especially when weathered.
- Weathering of the rock mass is ubiquitous and ranges in intensity from moderate to severe.
- The foliation in the schist generally strikes eastward and dips approximately 40 degrees southward parallel to the slope on which the pier is located.
- The schist contains well developed cross joints that strike northward and dip sub-vertically.
- Drilling results in borehole GA-3 indicate the rock beneath the pier is highly weathered and broken.

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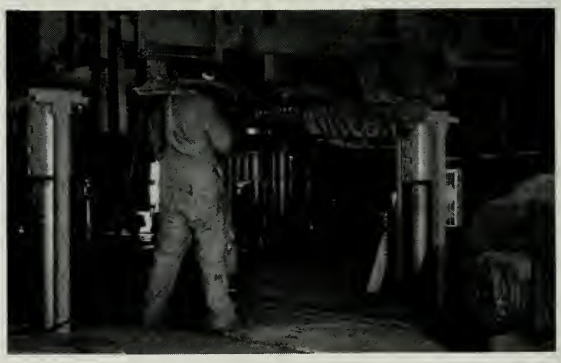

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Figure 7. Foliation planes within Birch Creek Schist.

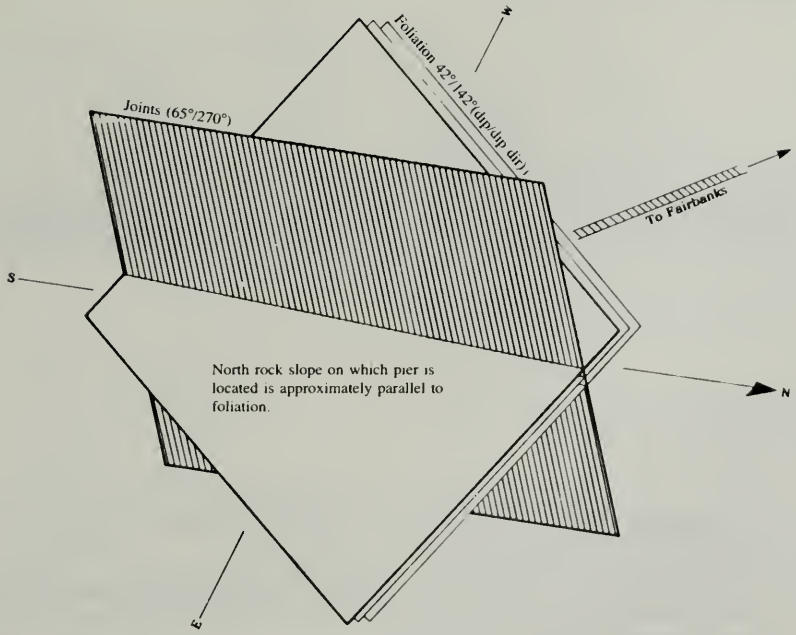

Figure 8. Isometric view of idealized geologic structure.

Failure mechanisms that were considered feasible included a shallow plane failure through the broken rock at or near the base of the pier and a deep-seated circular or bearing capacity failure through the rock mass as shown in Figure 9. For each of the feasible failure modes, stability analyses were undertaken to determine the existing stability conditions and the improvement to stability which could be effected by alternative remedial methods.

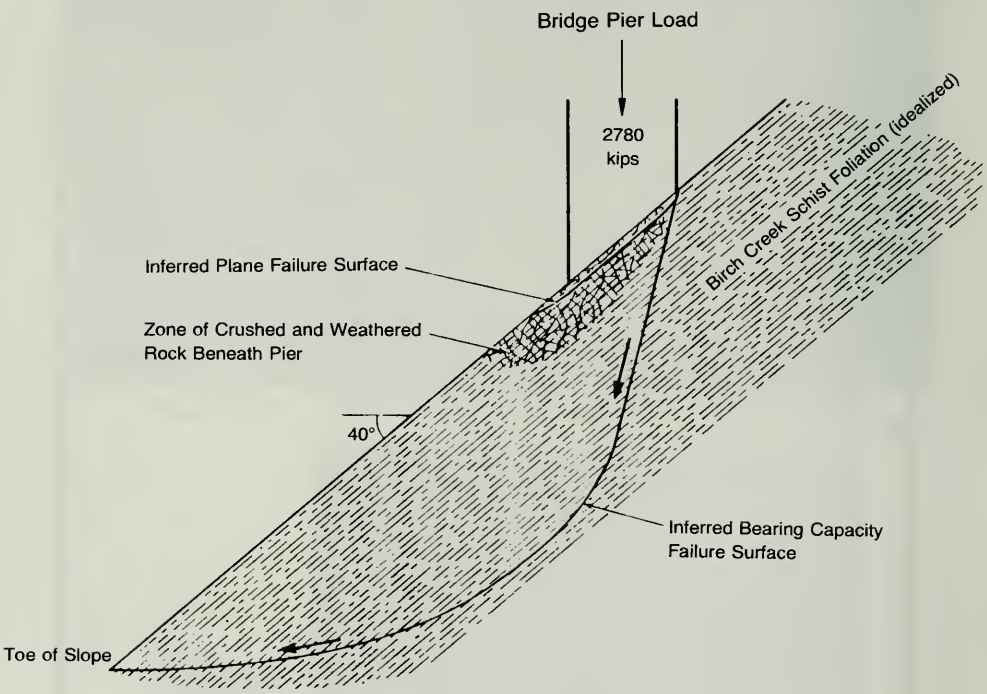

Figure 9. Potential failure mechanisms

STABILIZATION ANALYSES

Material Properties

The engineering property of primary interest for the analysis of pier stabilization alternatives is the shear strength of the rock mass and of discontinuities within the rock mass. In the absence of laboratory strength testing, engineering judgment was used to assign strength values to the rockmass in the directions parallel to and across the foliation. These strength values were validated by a process of back-analysis which confirmed that the existing slope and pier were unstable (i.e. Factor of Safety less than unity). The resulting shear strength parameters selected for engineering design within the Birch Creek schist at Bridge 355 are shown on Figure 12.

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External Factors

Stability calculations incorporated the maximum design loads for the pier which included dead, live and impact loads which totaled 2780 kips. Based on drilling results, the long-term phreatic surface was interpreted to be below potential failure surfaces and the slope drained for stability analyses. However, to account for worst-case high runoff conditions, the stability analyses also assumed that the phreatic surface was at the ground surface and the slope was therefore assumed to be saturated. A design factor of safety in the range of 1.5 to 2.0 was adopted in general accordance with Section 8.5.4 of the A.R.E.A. manual (1987) which applies to concrete structures and foundations.

Plane Failure

In this mechanism of failure, the assumed surface of movement was along a plane, possibly defined by foliation, through the crushed and weathered rock mass beneath the pier (Figure 9). The limit equilibrium analyses consisted of resolving forces acting at the base of the pier to determine the net unbalanced force acting in the downslope direction. Alternative stabilization methods which were analyzed to withstand this force included tensioned anchors and passive shear pins.

Initially, used rail was considered for use as shear pins. However, access and equipment restrictions to drilling a hole of sufficient diameter to accommodate the rail and grout precluded this option. As an alternative, shear pins consisting of number 18, grade 75 reinforcing bar with a nominal cross section of 4 square inches and a shear strength of 50,000 psi or 200 kips per bar were used. The analyses consisted of determining the sensitivity of the Factor of Safety to the number of shear pins installed.

Rock anchors with design tensile strengths ranging from 50 kips to 200 kips were considered. The lower strengths could be developed with deformed steel bars while multi-strand cables would be required for the higher strengths. The rock anchors were assumed to be installed in pairs.

Results of the plane failure analyses are shown in Figures 10 and 11. Using the assumed shear strength parameters for the rockmass, the calculated Factor of Safety for the pier in its current condition was 0.70. As shown, both shear pins and high capacity rock anchors were feasible to stabilize the pier against a base failure. However, the lower strength anchors would require an impractical number of bars and were therefore not feasible. Thus, if anchors were selected as the stabilization method, high capacity cables would be required. The drawback of these cables is that considerable bond length would be required to develop the necessary anchorage in the poor quality rock at the site.

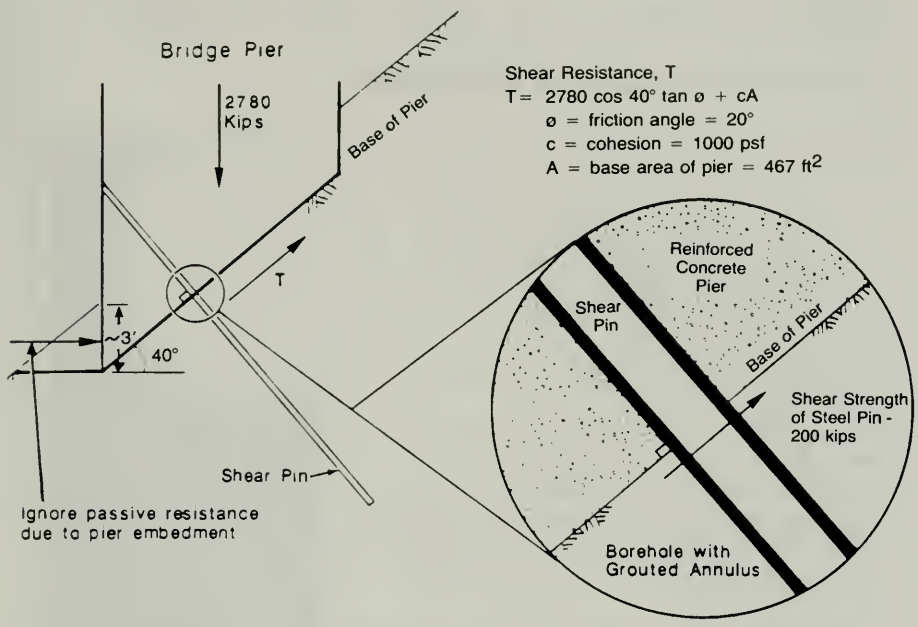
Circular-Bearing Capacity Failure

The circular or bearing capacity failure was analyzed using a limit equilibrium method developed by Sarma (1979). The shape of the critical failure surface was approximated by a spiral curve with its starting point at the upslope edge of the pier. This surface was divided into slices with edges radiating from the downslope edge of the pier (Figure 12). For each slice, shear strength parameters were assigned to the side and base in accordance with the intersection angle with the foliation or the presence of broken rock. Sarma's method of analysis determines the cumulative driving force and resisting force for all slices and thereby determines the Factor of Safety.

Three potential methods of providing stability against a circular or bearing capacity failure were analyzed; a fill buttress downslope of the pier, tensioned rock anchors and passive shear pins.

The calculated Factors of Safety for the current condition and for each of the stabilization alternatives are shown in Figure 12. Without support, the calculated Factor of Safety for the pier was 0.80. This was consistent with the long term creep movements which have been experienced and the fact that the bridge structure was probably applying restraint to the pier.

Both the tensioned rock anchors and passive shear pins were feasible to provide an adequate Factor of Safety against a deep-seated failure. However, in the case of rock anchors the high capacity



CASE 1: CURRENT CONDITION (no support)

F.S. = (2780 cos 40° tan 20° + 1.0 × 467)/2780 sin 40°
F.S. = 0.70

CASE 2: SHEAR PINS (#18 bar: 4 in^2 cross-section)

Shear strength per pin = 200 kips
F.S. = (2780 cos 40° tan 20° + 1.0 × 467 + N • 200)/2780 sin 40°
F.S. = (N + 6.21)/8.93

Number of Pins N	Calculated Factor of Safety F.S.
0	0.70
2	0.81
4	1.14
6	1.37
8	1.59
10	1.81

Figure 10. Plane failure analysis with shear pins

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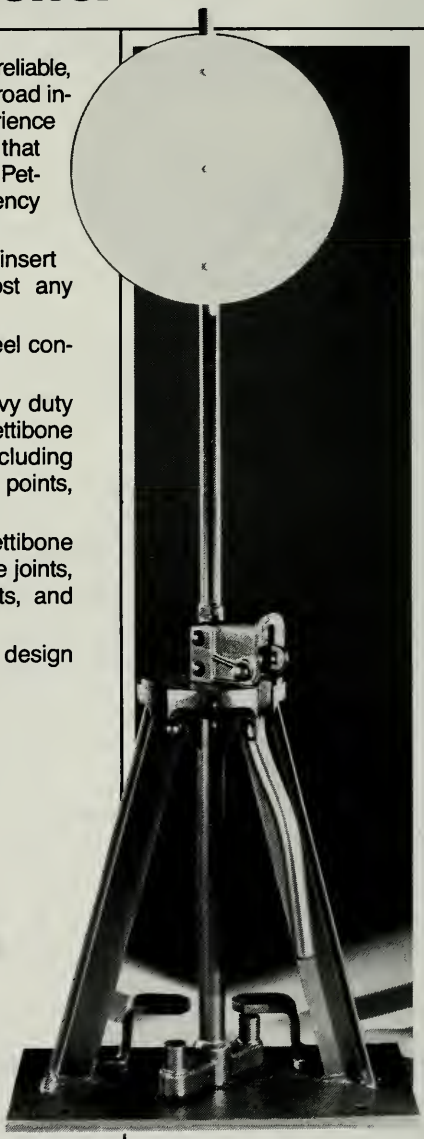

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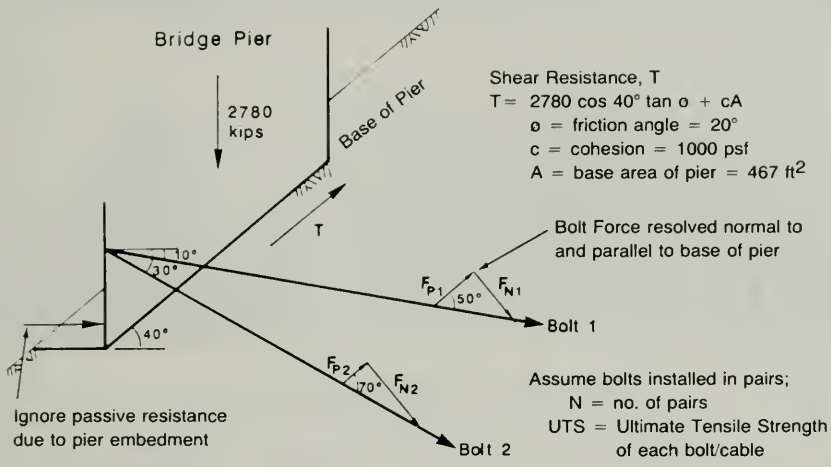


FACTOR OF SAFETY

F.S. = Resisting Forces/Driving Forces

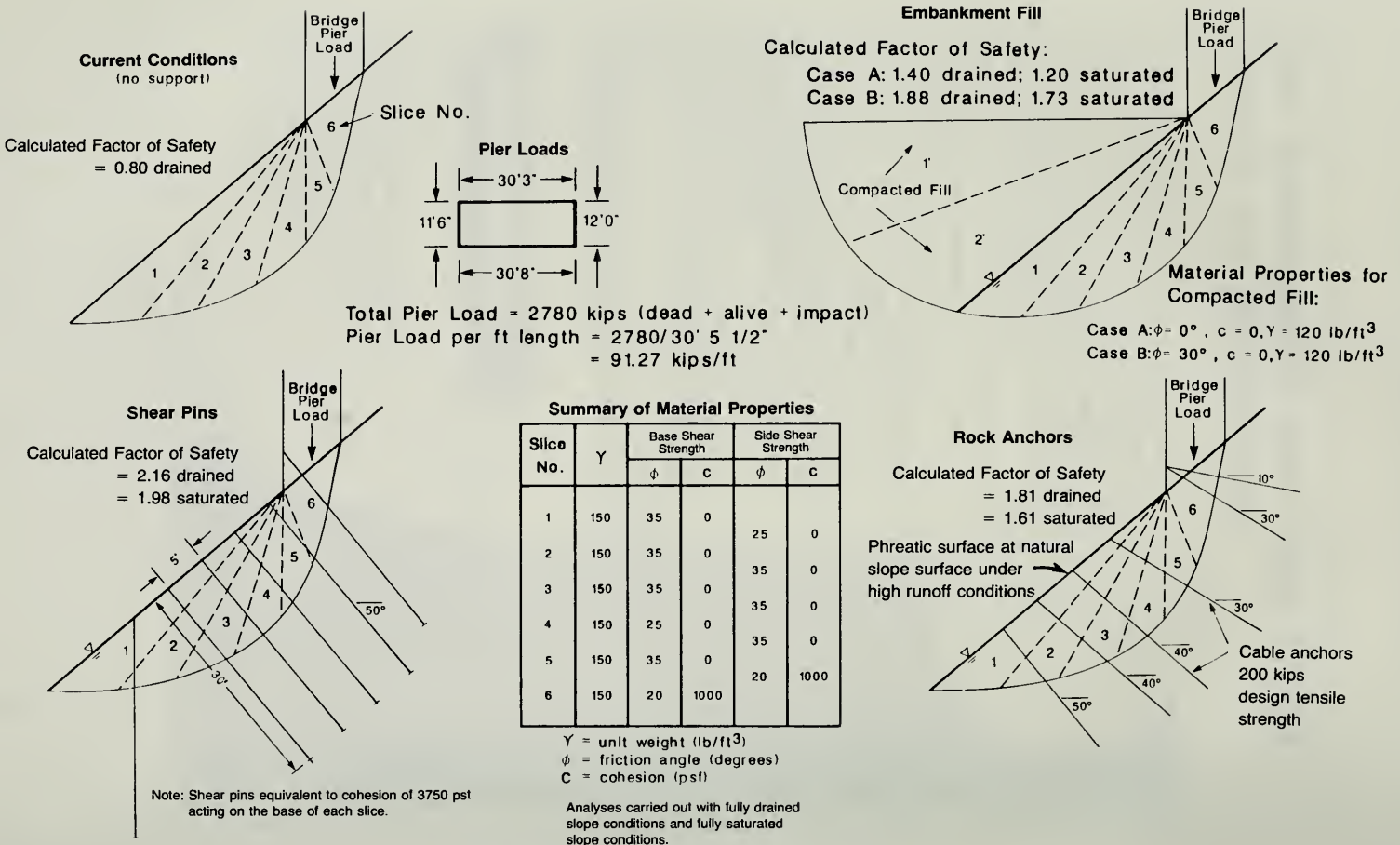
$= (T + F_{P_1} + F_{P_2})/2780 \sin 40°$

$= [(2780 \cos 40° + N \cdot UTS(\sin 50° + \sin 70°)) \tan 20° + 1.0 \times 467 + N \cdot UTS(\cos 50° + \cos 70°)]/2780 \sin 40°$

NO. OF BOLT PAIRS, N:

	CALCULATED FACTOR OF SAFETY			
	1.25	1.5	1.75	2.0
UTS= 50 kips	13	18	24	29
UTS= 100 kips	7	9	12	15
UTS= 200 kips	3	5	6	7

Figure 11. Plane failure analysis with rock anchors


Current Conditions
(no support)
Bridge Pier Load
Slice No.
Calculated Factor of Safety
= 0.80 drained
Pier Loads
30'3"
11'6"
12'0"
30'8"
Total Pier Load = 2780 kips (dead + alive + impact)
Pier Load per ft length = 2780/30' 5 1/2"
= 91.27 kips/ft
Embankment Fill
Calculated Factor of Safety:
Case A: 1.40 drained; 1.20 saturated
Case B: 1.88 drained; 1.73 saturated
Compacted Fill
Material Properties for Compacted Fill:
Case A: ϕ = 0°, c = 0, Y = 120 lb/ft3
Case B: ϕ = 30°, c = 0, Y = 120 lb/ft3
Shear Pins
Calculated Factor of Safety
= 2.16 drained
= 1.98 saturated
5'
50°
30°
Note: Shear pins equivalent to cohesion of 3750 psf acting on the base of each slice.
Rock Anchors
Calculated Factor of Safety
= 1.81 drained
= 1.61 saturated
Phreatic surface at natural slope surface under high runoff conditions
10°
30°
40°
50°
Cable anchors 200 kips design tensile strength

Summary of Material Properties

Slice No.	Y	Base Shear Strength ϕ	Base Shear Strength c	Side Shear Strength ϕ	Side Shear Strength c
1	150	35	0	25	0
2	150	35	0	35	0
3	150	35	0	35	0
4	150	25	0	35	0
5	150	35	0	20	1000
6	150	20	1000		

Y = unit weight (lb/ft3)
ϕ = friction angle (degrees)
C = cohesion (psf)

Analyses carried out with fully drained slope conditions and fully saturated slope conditions.

Figure 12. Summary of circular failure analyses

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multi-strand cables would be necessary to provide the required restraining force without an excessive number of anchor installations.

As shown in Figure 12, an embankment fill placed to a height equal to the base of the pier would provide a Factor of Safety between 1.40 and 1.88 depending on the assumption for the mobilized strength of the fill. In order to mobilize maximum strength, fills typically require a specific amount of displacement which the pier may not have tolerated without jeopardizing the bridge structure. Furthermore, the inability to rapidly mobilize the equipment necessary to replace the bridge with a fill precluded this alternative from further consideration.

ALTERNATIVE STABILIZATION METHODS

The stability analyses for the plane and bearing capacity failure modes indicated that the pier could be mechanically stabilized. The stabilization method to be selected clearly had to provide an adequate Factor of Safety for both the plane failure and bearing capacity mechanisms of failure. Based on economy of construction, material availability and technical superiority, the preferred alternative was to utilize fully-grouted passive steel pins acting in shear.

Passive Shear Pins

The use of untensioned steel dowels to stabilize large-scale rock slopes is a relatively new innovation derived from experience in open pit mining (see Rosengren (1986, 1987)). The basic design consists of steel shear pins grouted into drillholes which extend through the inferred failure surface or surfaces.

Passive shear pins mitigate against rock mass failure in two ways. Firstly, the technique utilizes the principle that in order for a rock mass to fail it must undergo dilation as the component blocks rotate to accommodate the movement. If this dilation can be inhibited, as with grouted steel pins, then failure will also be inhibited. Secondly, the cumulative cross-sectional area of all the steel pins penetrating the failure surface adds a significant strength component to the rock. This strength component is treated as a cohesion value equivalent to the shear strength of the steel across the failure surface (Figure 12).

DESIGN CONSIDERATIONS

Adhesion

The adhesion between the grout and the walls of the borehole is a required design parameter for the pins. This would normally be determined for the specific construction site through a series of pullout tests. Because this was not practical for Bridge 355 and due to the very broken nature of the rock at and below the pier, a conservative adhesion value of 3 tsf was selected.

Bond Length Design

Although the pins are designed to act in shear, a tensile force is transmitted by the pin to the rock beneath the failure surface during displacement. It is therefore necessary to determine the required bond length for that portion of the pin beneath the failure surface. For pins acting in shear, there is not an accepted rational method to calculate the required bond length. In the limiting case the bond length should be sufficient to resist the full tensile capacity of the steel pin. This was felt to be overly conservative and the decision was made to arbitrarily select a bond length of 20 feet beneath the failure surface. With the adhesion value of 6.3 kips/foot of bond this is equivalent to about one-third the ultimate tensile strength of #18 bar.

CONSTRUCTION

Equipment

Due to the importance of the bridge to coal shipping, it was mandatory that construction be accomplished under live track conditions. Apart from the financial incentive to carry out the stabilization work in a short period of time, there was a strong technical reason to have the work

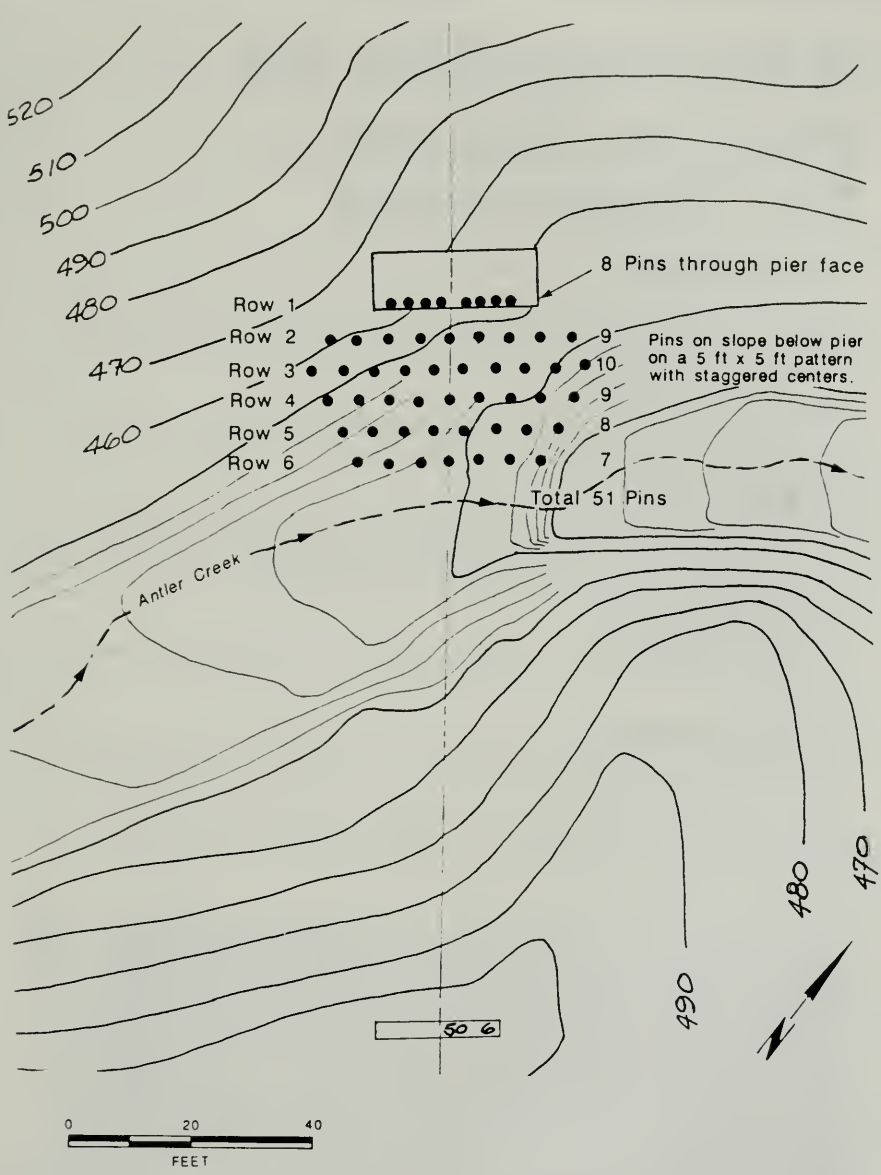

Figure 13. Recommended shear pin locations

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proceed quickly and without interruption. The latter concern was that if only a portion of the shear pin program was installed due to an unforeseen work disruption, ongoing movement could over-stress and fail the installed pins. These considerations, combined with the remote location of the site required that the logistics of the project be carefully planned and executed. Throughout the construction period, the ARRC provided a B&B work train on site to assist construction crews. Additional equipment provided by the railroad included a locomotive crane, a D-6 track dozer, an air-track drill rig and a grout plant.

Material Specifications

Shear pins consisted of #18 deformed steel bar, Grade 75, with a nominal diameter of 2.25 inches and a cross-sectional area of 4 square inches. The guaranteed ultimate tensile strength of this bar is 100 ksi.

The grout was Portland cement-based and developed a minimum compressive strength of 5000 psi at 28 days. The water-cement ratio of the grout was kept as low as possible to be compatible with pumping equipment. A maximum ratio of 0.5 water to cement by weight was specified.

Drillhole Locations and Orientation

A total of 51 shear pins were installed in six rows through the pier and on the slope beneath the pier. A nominal lateral spacing for the holes in each row was 5 feet so that each row would accommodate about 8 pins (Figure 13). The row spacing was about 5 feet or as close as access equipment would allow and pins on alternate rows were staggered. All holes were drilled normal to the existing slope face with the exception of the lowermost row which was drilled vertically. In this fashion the pins will be approximately perpendicular to the inferred failure surface(s).

Due to the very broken rock in the toe area of the pier and the potential for planar failure, the first row of eight evenly spaced pins was installed through the base of the pier (Figure 14).

Hole Length

All holes were drilled to 30 ft. depth. For those holes through the pier, the holes were located such that approximately the upper 10 ft. of the hole was in concrete in order to provide an adequate bond of the shear pin to the concrete pier.

Drilling Practice

Drill pads were constructed on fill to avoid undercutting the foliation of the schist and possibly inducing a slope failure. Care was taken that the bond between the grout and the rock at the hole perimeter would not be reduced by a mud cake. To this end, all drilling was accomplished using air-flush to remove cuttings.

Drilling advance rates were monitored to detect the presence of thick, soft zones within the rock mass. If zones in excess of one foot in thickness had been encountered, a construction contingency provided for possible additional shear pins or other appropriate measures. Standing water was not observed in the drill holes.

Shear Pin Installation

Centralizers were initially intended to be used to position the steel bar in the drill hole and thereby to provide a uniform grout annulus for corrosion protection of the steel. However, due to the inclination of the shear pin holes, coupled with the soft, weathered nature of the foundation rock, placement of the shear pins with centralizers was not possible. A tremie pipe was used to grout the hole annulus from the bottom up. Grouting pressures were kept low to avoid possible stability problems induced by "hydro-fracturing" of the rock. Although pressures did not exceed 50 psi, grout volume measurements indicated a 20 to 30 percent grout take into the formation rock surrounding the drillhole which was beneficial to improve the strength of the overall rock mass.

Figure 14. Drilling first row of shear pin holes angled at 45° through the base of the concrete pier.

Monitoring

Monitoring performed prior to construction indicated subtle movement, less than 0.25 inches, over a three day period. After placement of four shear pins in the first row through the pier, no further movement was detected. This observation correlates well with the analytical results.

Due to the lack of precedent for passive methods of slope reinforcement in this geological environment, recommendations were made for monthly displacement monitoring to be carried out to verify the remedial works. This monitoring was subject to increased frequency if movements were detected or excessively wet weather or rapid snowmelt conditions were experienced.

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RAILROAD PERSPECTIVE

The bridge at MP 355 was an impediment to rail traffic for a number of years prior to the catastrophic floods of October, 1986. Over the years the 355 bridge suffered the consequences of its alignment in rather spectacular fashion, with at least one derailment on the structure. The bridge had been shimmed and relined annually, with a total vertical displacement well in excess of a foot.

Following the 1986 floods, local runoff created a serious problem which exacerbated the settlement of the north pier. The vertical and lateral displacement suffered by the superstructure required the imposition of minimum train speeds and patrols ahead of every train.

The primary advantage to the railroad of the slope-pinning operation was that it corrected a serious threat in a relatively short period of time. The equipment was mobilized to the site by rail and set in place on the slope with a locomotive crane. The actual construction duration was only eight days and was carried out with minimal interference to train traffic.

The fact that locally available materials and railroad forces could be utilized in the remedial construction resulted in a significant cost saving to the railroad. The total cost for the slope stabilization project was $160,000 which included all engineering, railroad equipment and labor, drilling and material costs. A portion of this cost was borne by FEMA. The technical success and the cost-effectiveness of the project has prompted the ARRC to consider the same approach at similar locations on the system.

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COMMITTEE 7 — TIMBER STRUCTURES

Chairman: D. C. Meisner

Report of Subcommittee D-1-64

Subcommittee Chairman: R. W. Thompson, Jr.

STUDY OF IN-PLACE PRESERVATIVE TREATMENT OF TIMBER TRESTLES

This subcommittee submits as information the final report on the study of the assigned project.

In-place preservative treatment of timber trestles got its start on the railroads in July of 1953, when six ballasted deck trestles were treated for the Gulf, Mobile & Ohio Railroad. Since that beginning, many railroad trestles have been treated in-place, using a combination of pentachlorophenol (normally referred to as "penta") and a light oil or similar vehicle applied under pressure to a series of bored holes in the timber to be treated.

The A.A.R. research center made a "quantitative analysis of in-place treatment" during 1968 and early 1969, culminating with a thirty-seven page report number ER-86, dated May 1969. This analysis was based upon test specimens taken from 3 different bridges:

Bridge No. 1 — In place treatment December 1955 and January 1956 samples taken 11 years after treatment.

Bridge No. 2 — Samples take immediately after treating.

Bridge No. 3 — Samples take three months after treating.

The findings from the A.A.R. analysis from Bridge No. 1 indicated that retention of penta in the eleven year old test specimen was sufficiently above the threshold value of .12 pounds per cubic foot to control decay in timber above ground. The average of all 104 samples of Bridge No. 1 the eleven year in-place treated specimen was .328 pounds per cubic foot. See report ER-86 from the AAR for Bridges 2 and 3 or for more information on Bridge No. 1.

In January 1981, test specimens were removed from three piling of the former New York Central's approach trestle to the Wabash River Bridge 17 at Mt. Carmel, Illinois and sent to a private laboratory for analysis. All piling tested were Creosote-treated southern longleaf pine. The piling analyzed were in-place treated with Osmose patented 24-12 Pentachlorophenol with several carriers designed to carry the penta into the decayed and sound wood of the piles. The preservative was pumped into 3/8" dia. x 10 inch holes under an average of 150 lbs. pressure in 1958. No follow-up treatment had been performed in the intervening twenty-two years.

The specimens were analyzed in December 1981. The following is a tabulation of the findings from the nine samples analyzed, and diagram of how samples were cut.

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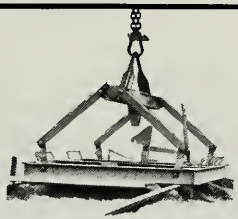

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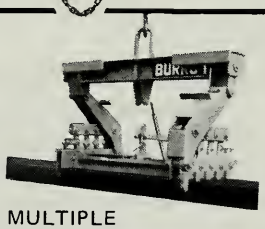

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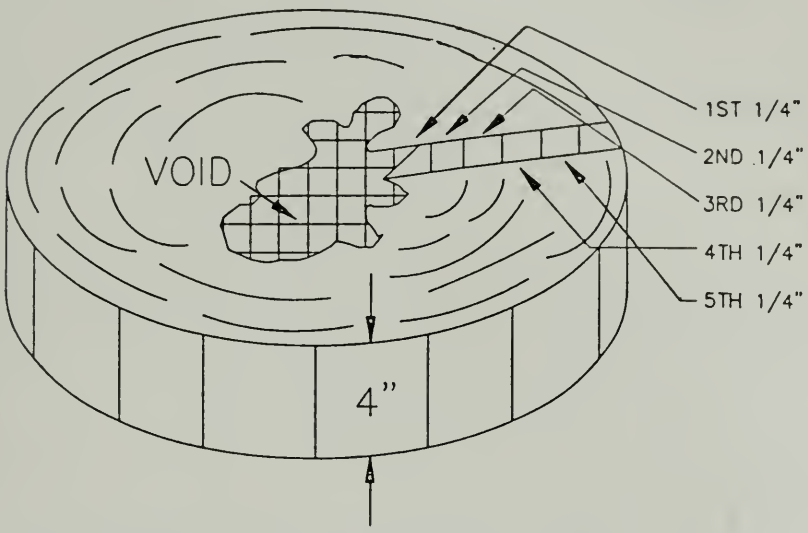

DIAGRAM OF HOW SAMPLES WERE CUT FOR ANALYSIS.

PENTACHLOROPHENOL
(Pounds per cubic foot)

Sample	*1st 1/4"	2nd 1/4"	3rd 1/4"	4th 1/4"	5th 1/4"
1A	.143	.114	.000	.000	.000
1B	.089	.079	.064	.000	.000
1C	.122	.102	.069	.072	.061
2A	.091	.080	.072	.080	.061
2B	.126	.118	.114	.099	.088
2C	.137	.137	.130	.126	.091
3A	.091	.084	.088	.076	.072
3B	.099	.053	.057	.000	.000
3C	.050	.000	.038	.000	.000

Borings were analyzed as per AWPA, A5 Section 6 the copper pyridine method for determining the retention.

(*The first 1/4" is taken adjacent to a point of internal decay or void and all measurements are outward from that point.)

Although, four of the nine samples show a penta retention slightly above the .12 pounds per cubic foot required for above ground control of decay, the analysis of these samples indicates that the effective life of the in-place treatment of bridge number 17 has been surpassed after twenty-two years.

Timber species, type of bridge, age of piling, geographic location, flooding cycles, quality of original treatment and other variables will contribute to the effective life of in-place treatment. The results of this study cannot be considered representative of all structures in-place treated since 1953, but this study indicates that decay in timber structures can be controlled and their service life extended by proper in-place treating.

Treating techniques, such as location of inspection and treating holes, have improved since the 1950's. New preservatives have been developed to be used with the original penta due to better knowledge of decay and EPA regulations. In the future, more new ways of controlling decay with environmental acceptable methods will have to be developed.

COMMITTEE 7 — TIMBER STRUCTURES

Chairman: D. C. Meisner

Report of Subcommittee D-2-74

Subcommittee Chairman: B. J. King

PROTECTION OF PILING AGAINST MARINE ORGANISMS BY MEANS OTHER THAN PRESERVATIVES

History

Historically marine borers have plagued man for centuries. Shipbuilders and waterfront structure owners have used various methods of protection for their property against the destruction by these minute animals, though with little success. About six years ago, the Navy estimated that over 500 million dollars in damage to waterfront structures is caused by marine borers annually in the United States alone. Considering the destruction occurring in other countries annually and recent inflation, today's losses could easily run into the billions of dollars.

These tiny marine creatures have long been damned and given many names. Fifteenth century records show that the Venetians called them Bissa, the Spaniards Broma, the Sicilians Beom, and the English Arters. Today, in France, they are called Taret, and in Australia, Cobra. Shipworms and Gribbles are also other accepted names. Christopher Columbus, Sir Francis Drake, and Captain James Cook all recorded complaints of damage by shipworms. Some historians believe that shipworm attack was a partial cause of the defeat of the Spanish Armada.

Egyptians began wrapping ship hulls with sheets of copper around 2,000 B.C. for protection which later led to the wrapping of piling around the year 40 A.D. In the 1700's, marine borers riddled the dikes that the Dutch had built to retain the sea. In doing so, they threatened the very life of Holland. This led the Dutch scientists to be the first to make extensive studies of marine borers. In more recent times, during the California gold rush of 1849, when several hundred square-rigged ships were in San Francisco harbor and surrounding areas at any one time, there sprang up innumerable small wooden structures to handle the freight and goods from these ships. By 1857, many of these structures were teetering on insecure foundations and had to be taken out of service. Those which remained had to be continually maintained by constant renewal of pilings. The attack progressed in some areas unchecked until freight and railway cars were plunged into the bay from the severely damaged wharves and ferry slips.

Marine Borers

The general term, marine borer, is used to designate several hundred species of marine invertebrates which bore into timber, soft stone and low strength concrete. There are two sub-families of these destructive organisms. which are more important to our concern, the Mollusca and the Crustacea. The molluscan borers have shells like that of the clam or oyster and the crustacean borers have an outer skeleton similar to lobsters or crabs. The mollusca sub-families include the genera; Teredo, Bankia, Lyrodus, Martesia, Hiata, and Pholads. The Crustacean sub-family genera include; Limnoria, Chelura, and Sphaeroma. Two of the Mollusca are of importance to our study, the Teredo and the Bankia which are the more common wood boring shipworms. Damage caused by the other genera of this sub-family is relatively inconsequential. Genera in the Crustacean sub-family which pose constant threats are the Limnoria and in some European and Pacific Island harbors, the Chelura.

Mollusks

Toredo and Bankia have similar physical characteristics, grayish in color with a shell on their head which is used for boring and a long worm like body which ranges in the adult from 3/8 inches (9.6 mm) in diameter and 5 or 6 inches (127 to 152 mm) in length with some species of Bankia growing to 3/4 inches (19 mm) in diameter and 48 inches (219 mm) in length. Two siphons are located on the posterior end of the body which is attached to the entrance hole in the wood. The siphons extend into the surrounding water and the incurrent siphon draws water with food in the form of diatoms and plankton

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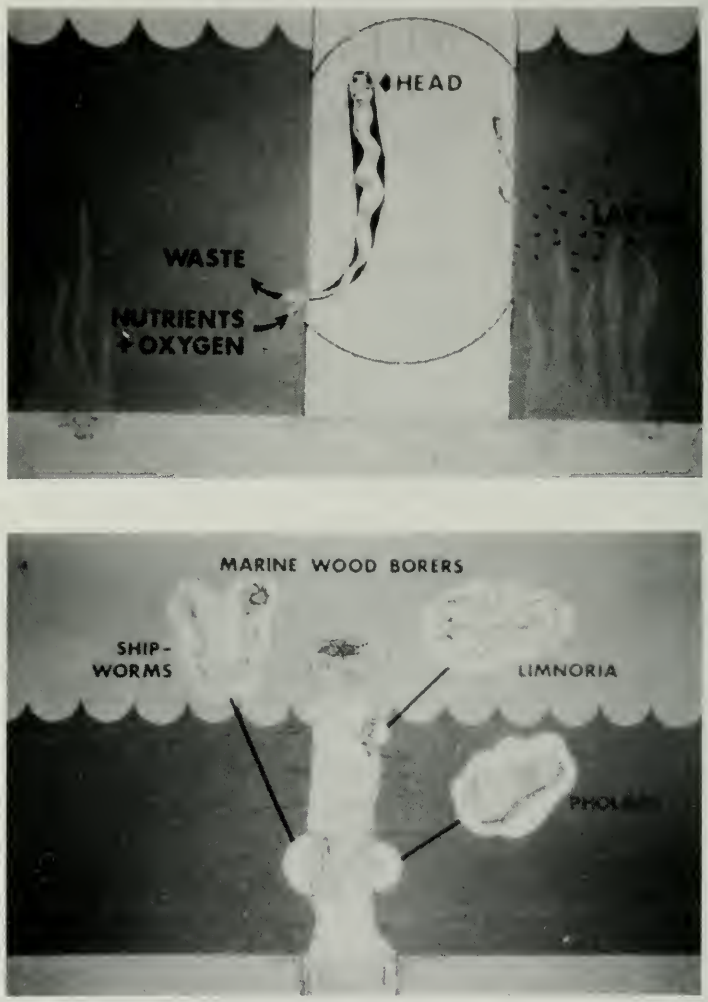
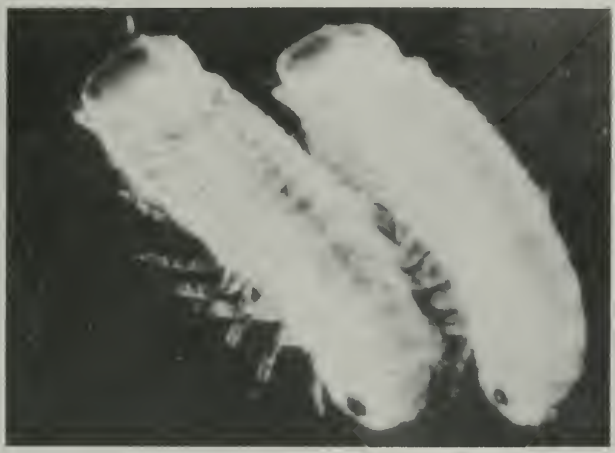


Marine wood borers

as well as dissolved oxygen for respiration. Waste products are expelled through the excurrent siphon and during breeding season, eggs are also passed by the female. The siphons may be retracted and the entrance hole plugged with a pair of small calcareous pallets which protect the borer from adverse external conditions. Eggs are carried by the currents and hatch into free-swimming, clam shaped organisms which attach themselves by a slender bypass thread to a wood surface. Changes in form occur and the clam shaped shell becomes modified into a efficient serrated cutting tool. Muscular contractions move the shell in a rasping motion which bores into the wood excavating a tunnel which becomes the borers home. Piling attacked by these so-called shipworms may appear completely normal on the exterior, though the interior may have a cross section similar to swiss cheese with the structural integrity of the pile completely gone.



Magnified Limnoria

Crustaceans

Crustacean borers, sometimes referred to as gribbles, have three genera important to our subject; Limnoria, Sphaeroma, and Chelura. Limnoria being the most important since it is the most widely distributed and dependent entirely upon wood for food and shelter. Sphaeroma and Chelura resemble Limnoria in appearance except they are larger, Sphaeroma averages over twice as large and Chelura, which are closely related to the shrimp, are more the same size as Limnoria only more cylindrical and pink in color.

Typical Limnoria attack damage. "Mae West" pile configuration in the attack zone.

Limnoria resemble the pill bug and are about 1/8 inches (3.2 mm) long and their width is approximately 1/3 their length. They have a small head with strong horny tipped mandibles with which they bore. Its body is segmented and has several pairs of walking legs fixed with sharp hook claws on the free segments and ending in a broad tail or telson. Limnoria are free swimming and destroy timber by producing a lacy network of tunnels at the woods' surface, never deeper than 1/2 inch (12.8 mm), which are continually eroded by wave action. They are known to attack from mean tide level to mud line. Heavy attack is common in the inter-tidal zone producing the familiar hour-glass contour effect on the deteriorated piling.

Among the several species of Limnoria, the Limnoria lignorium (Rathke), and Limnoria tripunctata (Menzies) are of major concern to our subject. Limnoria lignorium which has infected waters in cool to temperate climates around the world, and Limnoria tripunctata which can be found in

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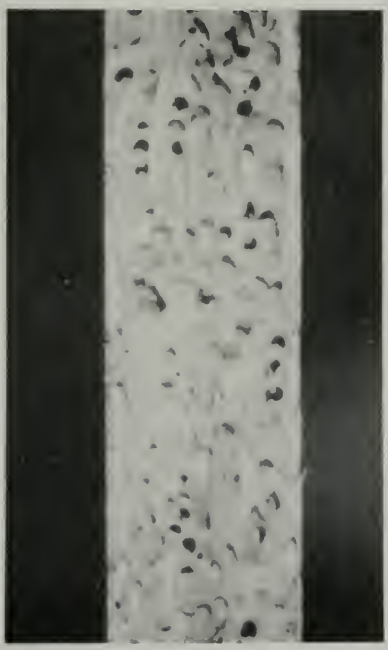
Magnified cutting shell at head of Teredo



Typical Teredo attack damage. Swiss cheese configuration of pile interior in the attack zone.

Bankia approximate 2 foot length

temperate to sub-tropic climates and which have developed a tolerance to creosote. Both species have exhibited tremendous destructive potential.

Preservative Treatment

In 1838, Englishman John Bethel invented and patented the process of pressure impregnating creosote into wood. This toxic creosote protection was put into use on Pacific Coast wharf structural piling in 1869 and was found to be a strong deterrent to the borers. The Robins' patent process developed later and more predominantly used consisted of treating piling with condensed vapors of creosote, injecting preservative amounting to about one pound per cubic foot. About this same time it was noted that piles driven with their bark on resisted borer attack better than those without bark. The protection afforded by the light retention of the Robins' process was poor and it became a more general practice for many years to drive the piling with the bark on. This was probably the earliest form of pile jacket or wrapping in modern times. During 1889, the Southern Pacific Transportation Company treated about one thousand piles at it's creosoting plant in San Pedro with an improved method, the injection of creosote being about fourteen pounds per cubic foot. These pilings were driven in Oakland Long Wharf and remained in excellent condition until 1919 when the Wharf was removed. Many of the piles were removed and redrive elsewhere, this being one of the earliest recorded successful attempts at protecting piling from marine borers.

Creosote and creosote solutions have been used for nearly 150 years and have performed very effectively, especially against Teredine attack. It has been less of a deterent to Limnoria, especially the tripunctata variety which seems to have developed a tolerance to the creosote. Waterborne preservatives comprised of formulations of arsenates of copper, which are quite stable when impregnated in the wood, are very effective in the prevention of attack by the Limnoria. The method of dual-treatment has since evolved into a treatment which involves an initial treatment with a waterborne salt preservative, followed by a creosote treatment resulting in an effective protection against both the Teredine and Limnoria.

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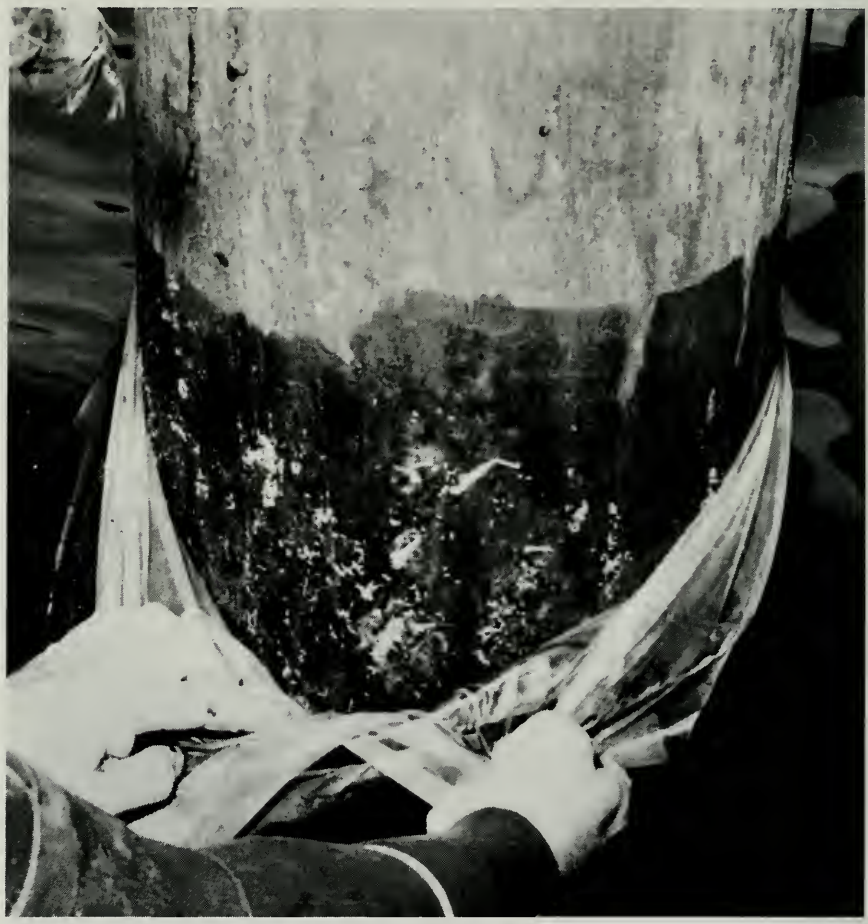

Flexible PVC wrap with polyethelene inner wrap to protect PVC from creosote in pile. After 24 hours Limnoria and all other organisms die from oxygen atmosphere.

Other Protection

The protection of piling against marine organisms by means other than preservatives reaches back to the Egyptians when sheets of copper were used to protect the hulls of ships. Innumerable plans have been developed from the most intricate to simply leaving the bark on the pile which flourished in the 1800's. A few of the various methods used and documented by harbor authorities are here cited.

Metallic barriers of various materials and alloys have been used, however, whenever a metal was used, it followed that the fastening materials must be of a composition which would minimize electromotive potentials. Although the use of copper for sheathing hulls and protecting underwater structures is historical, it is found that copper is not sufficiently durable in sea water to warrant its use. It will only last approximately 15 years for a 30 mil thick wrap and often the copper nail will fail before the jacket is perforated. Other corrosion resistant nonferrous metals, such as aluminum, cupro-nickel

alloys, etc. have been used as sheathing usually being secured with nails of the same metal. The nails have proven effective unless attacked by erosion or mechanical abrasion. Spot repairs may be made by using the same metal as a patching material. Metal barriers are expensive due to the high initial cost of the corrosion resistant materials and the difficulty to install and maintain.

Non-metallic barriers, such as fabrics impregnated with preservatives, various hydrocarbons or plastics have been used by winding around or affixing to the pile by means of wood battens. Other non-metallics were troweled on the pile above or below water. The maintenance of the integrity of these barriers, especially if applied before driving is usually difficult and the high cost of both materials and installation objectionable.

Semi-rigid nylon material show good longevity but has prohibitive material costs. This was found to be the case with polyester impregnated fiberglass. Rigid polyvinylchloride, another non-flexible non-metallic has performed well but the material cost is high and it should be considered only where its high impact strength is required. Coal tar impregnated rigid fiberboard encountered problems of splitting spirally and Limnoria penetrated the coal tar impregnated tubes.

The use of concrete as a pile protective jacketing material goes back to antiquity. When properly installed in the optimum conditions and manner, and providing bending does not become a factor, they are durable and do a very adequate job. Certain methods of applying a concrete jacket, with reinforcing included, can renew the structural capability of the piling. There are several methods of applying the concrete such as tremie placed in removable forms, grout pumped into aggregate prepacked forms, shotcrete on reinforcing mesh at low tides or before pile driving, poured in sonotubes or oil drums forms, and pumped into wire reinforced nylon bag forms.

The latter is probably the most commercially utilized method at this time. This underwater concrete-forming system utilizes high-strength, zippered, nylon fabric sleeves. These pile jackets are tailored to the exact length and diameter required and concrete mortar is pumped into the water permeable forms. The installation is relatively fast and easy which leads to economy.

The Port of Los Angeles Testing Laboratory has had an ongoing program for over 40 years, testing various methods and materials for the protection of piling against marine organisms by means other than preservatives. They began field testing of sheet plastics as a barrier material in 1956. Following long exposures of a number of sheet plastics in waters where marine borers were known to be active, polyvinylchloride (pvc) was tentatively selected as an excellent material for several reasons: during the test period, it was not adversely affected by marine organisms or sea water, it could be procured in adequate widths and thicknesses, it is tough, unaffected by floating oils, relatively stable to the action of actintic rays, and barnacle growths do not penetrate the film. Several schemes of application have been developed, actual tests were made at Los Angeles harbor on the helical wrap, zipper fasteners and heat-sealing. It was found that helical wrapping must have the lap sealed because barnacles can lift the edge of the overlap and permit Limnoria to enter. The zippers had a fairly high rate of failures and it is very difficult to achieve good conformity to the pile surface without additional ties. An envelope of PVC prepared by heat-sealing using an instantaneous electronic plastic welding machine to seal the lap along the length of the pile worked successfully. However, the envelope must be placed on the pile before driving and held in place by straps during the driving operation. This method is restricted to new piling. All three of the above methods were found to be expensive and undependable. Long-term use of the PVC wrap on creosoted piling revealed that its exposure to the creosote will harden and brittle it causing early failure. An inexpensive underjacket of thin polyethelene which is completely resistant to creosote is recommended and common practice. The Port of Los Angeles is currently doing extensive testing and wrapping of new piling with heat shrunk polyethelene wraps.

In 1956, Orval E. Liddel, a Civil Engineer of Avalon, California, with the assistance of Carrol M. Wakeman, Testing Engineer for the Port of Los Angeles, devised an economical way to encapsulate in-place piles with polyvinylchloride wraps by means of pre-attached pole pieces. This simple installation was conducted by encircling the piles with barrier material in convenient modular lengths,

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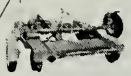

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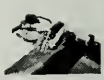

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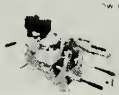

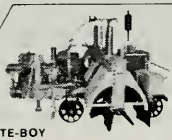

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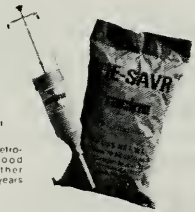

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rolling up the excess material on the pole pieces with a ratchet wrench to produce a tight form fitting barrier. The barrier wrap was nailed through the poling piece and sealed top and bottom with a monel strapping material and nails. This wrap creates a stagnated environment within and in a matter of a few hours, the dissolved oxygen content of the water drops to zero, lethal to existing borer life and preventing any new attack, all accomplished within hours by the pliable cover. Mr. Liddel secured a basic patent on the application procedures and it has been utilized around the world. The application has also been adapted for steel piling, both "H" and round in section, utilizing the zero oxygen environment to preclude the steel from oxidizing or rusting.

Summary

In summation, the Committee acknowledges that it has only touched on the breadth and depth of this subject, however, with attainability, effectiveness and economics as criterion, our opinion is that the following remedial methods represent state of the art protection of piling against marine organisms, by means other than preservatives.

1. In-place pile protection with flexible polyvinylchloride installed by the Orval Liddel Patented Method.
2. In-place pile protection in high impact areas by rigid polyvinylchloride.
3. In-place pile protection and structural reinforcement by concrete placement in nylon bag forms.
4. Preplacement pile protection by heat shrunk polyethelene envelopes.

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COMMITTEE 16 — ECONOMICS OF PLANT, EQUIPMENT AND OPERATIONS

Chairman: C. Bach
Subcommittee Chairman: H. C. Petersen

ECONOMICS OF TRAIN SPEED

Preface

Train operating speed can have a major impact on railroad economics by affecting both service to shippers and the physical plant of the railroad. To some degree, these provide opposite incentives for changes in train speed: while it is recognized that higher speeds can increase the demand for rail transportation (and thus railroad income), increased operating speeds generally result in higher expenses for track and equipment maintenance, and increased fuel consumption. It thus becomes necessary to try to find an optimal balance for each type of train.

The economics of train speed, with its engineering and marketing implications, is increasing in importance and effect on railroad profits and operations. Decisions as to operating speed must involve consideration of many factors such as additional revenue, market demands vs operating and maintenance costs, and equipment and engineering costs.

In many cases, a systems optimization approach can be expected to indicate an increase in operating speeds, at least for some types of trains and traffic. New opportunities exist for railroads to increase traffic through integration of rail services with industrial operations. Deregulation has opened opportunities for railroads to provide flexible, shipper-oriented service on a contract basis. Industries are installing new automated material handling equipment, and are increasingly recognizing the importance of manufacturing logistics (which includes transportation in the production system) to improving productivity. By providing time-sensitive railroad operations which can be integrated with industries and their sophisticated material handling systems, railroads may be able to increase their market share of transportation, especially of higher valued (and rated) shipments. But without careful analysis, operating speeds and associated costs over a given track segment may be increased beyond the level of an economic return, or yard delays may cancel any gains obtained by the higher operating speeds. Schedule reliability may be more important than speed. Thus any examination of the economics of train speed must include an evaluation of all phases of the route and the service required, and not simply maximum freight train speed over an isolated section of the railroad.

Effects of Operating Speed and Yard Delays on Car Schedule Time

The purpose of railroad operations is to effectively move rail carloads from origin to destination, which is usually accomplished by assembling the cars into trains. The rationale for an increased operating speed is to decrease total transit time. If enough loaded cars exist at one time for unit train operation, car scheduling and train scheduling can be the same. Otherwise, individual cars and blocks of cars will be combined in a train, and may be yarded a number of times en route; in this case the total delay to each car will be the sum of all delays, including pickup, yarding, car repair, and setout delays in addition to delays due to slow train operation. Nevertheless, other delays being equal, faster trains result in shorter car schedules.

There are three philosophical approaches to determination of operating speeds for freight trains. The first is the drag freight philosophy. Often used for high-tonnage bulk shipments such as coal or crushed limestone, this approach involves maximizing trailing tonnage per locomotive unit through low speed operations, possible 30 miles per hour or even less.

The second, time-freight approach, at the other extreme from the drag freight philosophy, involves operation of high-horsepower freight trains at speeds of 70 miles per hour or more. If combined with rapid yarding and delivery at each end of the trip, such as is done by some intermodal operations, the high train speeds may result in significantly shorter in-transit times and perhaps increased business. But if yard delays negate the time saved by high-speed operation, the extra speed-generated costs have been largely wasted.

The third approach balances the effects of operating speed on both market demands and operating costs, within the context of existing yards and schedules, and with adequate concern for car schedules. Operating speeds faster than the drag freight will usually be employed to attract traffic (and possibly to avoid track and equipment damage caused by resonant rock-and-roll), but the higher-speed schedules will be integrated with improved yarding, blocking, and delivery to avoid requiring cars to "hurry up and wait" with little overall gain in car schedule time. Little is to be gained by doubling a 30 mph speed to 60 mph over a 300-mile long line haul, saving five hours, if the car must then wait 24 to 48 hours before departing the yard; improvements resulting in yard delay reduction would have a greater impact on car schedule time, at possibly lower costs.

In many cases, this third approach will specify different speeds for different types of freight, employing some elements of both the drag freight and the time freight philosophies. Care must be taken when operating mixed speeds, however, to avoid track-damaging dynamics and forces, particularly on curves superelevated for higher speeds, where slow heavy cars can cause excessive damage to the inside rail. Adequate sidings and dispatching procedures must be employed to minimize delays; new computerized CTC (centralized traffic control) decision support dispatching systems can be helpful here. A clearly-stated policy should be developed to avoid uninformed field decisions as to train consist. Properly applied with adequate planning, modeling, and analysis, the balanced systems approach is the best way to set train speeds in a manner which optimizes car scheduling without incurring excessive costs.

Alternatives to Increased Train Speed

Prior to increasing train speeds to reduce car schedule time, alternate methods should be considered. Some of these alternatives to increased train speed may produce other benefits as well, such as reduced loss and damage, decreased car-hire expenses, reduction of yard congestion and yard crew requirements, and more consistent car scheduling and equipment utilization. In some cases, attention to these alternatives will yield the desired service improvements, eliminating the need for increased train speed and its associated costs. In other cases, these alternatives will yield further improvements when combined with speed increases. Simulation of the overall system may help to analyze the effects of these multiple improvements.

Alternatives to increased train speed include:

1. Use of run-through trains to bypass classification yards;
2. Pre-blocking and block-switching cars;
3. Improved dispatching/control systems to minimize delays due to train meets;
4. Coordination of connecting freight train schedules;
5. Reduction of car pickup and delivery times; and
6. New transport concepts such as roadrailers and integral trains.

Projected Traffic

An important part of the evaluation of train speed economics is an examination of the impact of train speeds on projected traffic. Along with other benefits to be gained from increased operating speeds, such as improved equipment utilization, one of the primary benefits may be the increased tonnage and/or rates which can be attracted through shorter in-transit times. This increase in demand due to reductions of in-transit times is called time elasticity.

Other traffic will not be expected to respond significantly to decreased train times. In most cases of bulk, lower-value freight such as aggregates, there is little or no time elasticity. Relatively lower speeds are satisfactory and will save the railroad expenses through reduced track investment, track and equipment maintenance, and fuel consumption, but will result in little if any loss of this bulk traffic. As long as rock-and-roll resonance is avoided, the primary determinants of bulk cargo train speeds may be operational, such as crew and equipment utilization, and track capacity and maintenance.

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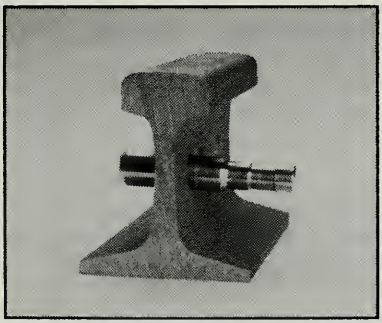

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Much of the higher-value traffic is time-sensitive with higher time elasticity. A substantial proportion of this traffic which has been lost to other modes (especially trucks) was lost due to long and unreliable in-transit times, not due to rates. Some of this type of traffic may be recaptured by increased train speeds, if other delays are also reduced or eliminated, especially if reliabilities of schedule times can be improved. Lower rates alone are often inadequate. Use of new-concept intermodal trains operating on tight, high-speed schedules with rapid service at terminals can recapture some of the freight which was lost to trucks. Some railroads have found, however, that inadequate intermodal traffic was attracted by faster service when a surcharge was imposed for that service.

Deregulation has provided an opportunity for railroads to contract with shippers to provide customized service. This service may be designed to interface with shippers' distribution and manufacturing logistics systems, which may involve specialized equipment and operations, Just-In-Time deliveries, and computerized controls and information exchange. While such agreements will usually require reasonable train speeds to reduce time and therefore in-transit inventory, many shippers will also place great emphasis on other aspects of rail service, such as loss-and-damage prevention and schedule reliability, as in the agreement between Conrail and General Motors for Just-In-Time delivery of auto parts.

In many manufacturing/distribution systems the shipment which arrives too early can cause the industry as much trouble as one which arrives too late. In these cases, operating trains on realistic, achievable schedules may be the best method of providing the desired dependability, even with slightly longer in-transit times.

Thus, market requirements should be analyzed as an integral part of an analysis of the economics of train speed. The final decision will require coordination between the engineering, operating mechanical, and marketing departments, as well as finance if capital investments are involved.

Benefits of Increased Speed

Increased train speeds which result in improved service (with higher rates in a few cases) can help to attract higher time-value traffic and new contract agreements with shippers, but other economic and operational benefits are also possible:

1. Locomotive fleet size reduction due to improved utilization. Shorter turn-around time may reduce the number of units required.
2. Possible increased track utilization.
3. Reduced crew costs, in some cases.
4. Improved freight car utilization, reduced fleet size, and lower per diem payments;
5. Possible longer time "windows" for track maintenance, resulting in greater maintenance-of-way productivity;
6. Intermodal/TOFC can offer more competitive schedules; and
7. Trains may be more easily paced toward meets and yards, increasing operational flexibility and reducing yard congestion.

Costs of Increased Train Speed

Increases in train operating speeds result in additional costs which can, in some cases, completely offset any economic gains. Thus, these costs and their impact on both short-term and long-term profitability of the railroad must be carefully analyzed. Increased train operating speeds can result in higher costs in the following areas:

I. Increased speed requires a higher quality of track geometry and maintenance standards. Higher standards by regulation require more expensive track construction and maintenance. Section 22, Part 4 of the AREA Manual for Railway Engineering suggests that track maintenance costs for a 60 mph speed can be increased 15% over 40 mph maintenance costs;

2. Greater forces can result from higher speeds, leading to more rapid track degradation, as well as car and locomotive equipment and maintenance costs. The increases in these costs will depend to some degree on the level of maintenance; for example, the effects of dynamic forces will be greater for poorly-maintained track;
3. Higher speeds can increase the severity of derailment damage. Not only are dynamic forces often higher, but the kinetic energy to be dissipated increases as the square of train speed;
4. Fuel consumption will increase due to mechanical drag (proportional to the speed) and aerodynamic drag (proportional to the square of the speed — see the modified Davis equation), but this can be at least partly offset through the use of greater train momentum on rise and fall profiles, and through reduced overall operating time. Train performance simulation programs can be used to estimate the effects for an actual operation;
5. Engineering (geometric) changes may be required, such as longer and/or more sidings, changes in curvature and superelevation, higher-speed turnouts, and possibly major line changes. Capital costs and disruption during construction must be considered and compared with the resulting savings due to the improvements;
6. If operating speeds of only some trains are increased, greater variability in train speeds and composition will probably increase equipment and track maintenance requirements and costs;
7. More locomotive units will be required for a given tonnage due to higher horsepower requirements, and different speeds may require units with different gear ratios, reducing compatibility;
8. Increased train speeds require greater spacing between signals, which may result in costly signal changes.

Other Factors Affected by Increased Train Operating Speed

Some factors which can be affected by increased train speed might be improved or made worse, depending on the existing schedules and operating characteristics of the segment of railroad affected. These must be analyzed, and the impact determined. Such factors include:

1. Yard congestion may be eased by "spreading out" arrivals and departures, or it may be increased if the schedules become "bunched up" with new emphasis on rapid throughput;
2. Intermediate yarding may be bypassed, but this could make the intermediate yard less cost-effective for local work;
3. Road crew costs could be reduced with less recrewing due to improved hours of service limitations, and crew scheduling could be improved with pool crews. For assigned crews, higher speeds could result in longer layovers which could increase crew costs. This is an area which requires careful analysis, as the economic impact can be large.

Balancing Benefits and Costs

Decisions as to optimal train operating speeds must be carefully analyzed using a systems approach, as these decisions will have a major impact on net operating income by affecting both service to shippers and the physical plant of the railroad. While faster service can often increase the demand for rail transportation, increased operating speeds generally result in higher costs for fuel, and track and equipment maintenance, as indicated in the previous sections. An operating speed for each type of train should be found that optimizes the complete system.

Economics of train speed cannot be optimized in a piece-meal fashion; each component of the total railroad system must be optimized as it affects the entire system profitability, not in a vacuum as individual components. Modern systems optimization techniques such as simulation and dynamic programming might be employed, along with appropriate models of individual subsystems such as plant investment, track maintenance, fuel consumption, market elasticity and demands, and mechanics of operation. Use of these aids requires accurate data and careful analysis; aggregated, oversimplified numbers will generally prove inadequate. The requirement for accurate numerical models and detailed data bases usually indicates that computers should be employed.

In some cases, alternative actions such as reduction of yard delays can have a much greater impact on service than increases in train speed, at a much lower cost, and should be considered in the overall systems study. A combination of improvements which may include changes in train speed along with other factors will often be found to be most cost-effective.

Once the models and analytical techniques have been programmed and used, they should be kept "on-line" for use in follow-up studies, both to evaluate and calibrate the models, and to take into consideration any changes which have occurred since the last evaluation. An ideal system would involve a continuing evaluation of real-time data, enabling rapid changes in operating policies to meet the changing economics of a dynamic transportation industry.

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Memoir

Lewis K. Sillcox

Dr. Sillcox, a member of Committee 16 for 50 years and AREA member for 68 years, died on March 3, 1989 of pneumonia at Watertown, New York. He was 102 years old and lived in Watertown.

Born in Germantown, Pennsylvania, on April 30, 1886, he married Edna May Harris on September 14, 1918. He is survived by a son, 4 daughters, 21 grandchildren and 22 great-grandchildren.

Dr. Sillcox received his education at Trinity, Schenectady, New York; Ecole Polytechnique of Brussels, Belgium (1903); Clarkson College of Technology, Potsdam, N.Y. (D.Sc 1932); Cumberland University (D. Eng. 1941); Syracuse University (LLD 1948); Purdue University (D. Eng. 1951).

His early career was spent in railroad and manufacturing organizations, including: 1903-07 as an apprentice with the New York and Hudson Railroad at High Bridge, N.Y.; 1907-09, as a foundry apprentice with McSherry Manufacturing Company, Middletown, Ohio; 1909-12, as shop engineer with Canadian Car and Foundry Co., Turcot Works in Montreal, Quebec, Canada; 1912-16, as a mechanical engineer with Canadian Northern Railway, Toronto, Ontario, Canada; 1916-18, as a mechanical engineer with Illinois Central, Chicago; 1918-27, as general superintendent of motive power with Chicago Milwaukee St. Paul and Pacific, Chicago.

In 1927, Dr. Sillcox began his career with New York Air Brake Co. in Watertown, New York as assistant to the president and was in charge of operating the company plant. He was largely instrumental for developing a new type of brake for automobiles. In 1929, he was appointed vice-president, as well as being the general manager and chief engineer of the plant. He became the first vice-president in 1945, and later executive vice-president in 1948. When New York Air Brake acquired control of Kinney Manufacturing Company of Boston, Massachusetts, he was made vice-president and director of that company. He was appointed vice-chairman of the board of New York Air Brake Co. with headquarters in New York in 1952 and honorary vice-chairman of the board in 1953. He also served as vice-president and director of the Hydraulic Equipment Co. in Cleveland, Ohio and Hydraulic Controls Inc. of Chicago, Illinois, both of which were subsidiaries of New York Air Brake.

He was selected by the Governor of New York, Nelson A. Rockefeller to be the first Director of Transportation when this position was created in 1958 by the Legislature. He was appointed to this position in 1959 when he retired from New York Air Brake. In addition, he was made a member of the New York-New Jersey Transportation Agency. He held this position till 1960.

He served in many executive positions in professional groups and schools of education, including chairman of the Mechanical Division A.A.R. (1926-27); Federal Co-ordinator of Transportation (1934-36); Chairman Mechanical Advisory Committee, School of Engineering, Princeton University; Chairman Metals Engineering Handbook Advisory Board of A.S.M.E.; Chairman Papers Committee, 8th Pan-American Railway Congress and Life Member of the Board of Trustees, Clarkson College of Technology.

He presented many papers and lectured at schools including Clarkson College of Technology, Norwich University, Purdue University, Northwestern University, Harvard Graduate School of Business, Yale University, MIT and Princeton University. He was the recipient of ASME Gold Medal (1943) and Institute of Locomotive Engineers (London) Gold Medal (1942). He was author of "Mastering Momentum" (1941) which was revised in 1946 and 1953. He contributed to many technical publications.

He attended Committee 16 meetings until he was 95 and reviewed Committee 16 reports up until he was 98. Committee 16 members will always cherish his talks at our meetings on what we, as active railroaders, should be doing to improve operations to ensure that railways remain competitive. He would often emphasize to us that "Time Was Money." On his 100th birthday, Committee 16 presented him a plaque recognizing his many years of distinguished service to AREA.

Con Bach

Errata for Article on "The Evolution and Application of Rail Profile Grinding," AREA Bulletin 718, Volume 89, December 1988

1. Page 473, last Paragraph, change " . . . field side of the rail head *to avoid* any false flange . . . " to " . . . field side of the rail head *avoiding* any false flange . . . ".

2. Page 477, Control of Short Wave Corrugations, last complete paragraph, change "0.6 inches, (4)" to "0.060 inches, (4))".

3. Page 480, Paragraph 5, change " . . . fatigue shelling, by wearing away the surface fatigue . . . " to " . . . fatigue shelling, by *artificially* wearing away the surface fatigue . . . ".

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Cover photo — Chicago and Northwestern coal line under construction north of Torrington, Wyoming, 1984.

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Photo 1 — British Columbia Railway Tumbler Ridge line under construction in 1983.

New Rail Line Construction in the 1980's

As the 1980's draw to a close it is impressive to review the amount of new-line construction in this ten-year period in North America. The 1980's just missed including completion of the final 85-mile section of the Gillette-Orin, Wyoming line of the Burlington Northern and the 128-mile Corondiro-Lazaro Cardenas line of the National Railways of Mexico, both of which were opened in late 1979. New Alignments for existing routes led to massive projects in Canada (New line through Rogers Pass) and Mexico (New 200-mile long high speed Mexico City-Irapuato Line and other projects, including a new 410-foot high double-track bridge on a line revision between Mexico City and Vera Cruz, as well as 210 miles of new lines to shorten the Guadalajara-Monterrey route by almost 100 miles).

Photo 2 — Construction of Washington Metro double-track transit line to Greenbelt, Maryland at point where it goes from subway to at-grade track, 1988.

Photo 3 — San Diego's rail transit system, shown here shortly after its 1981 opening, was the first in the U.S. of a new breed of "light-rail" transit lines.

In addition to numerous new lines built to serve individual new mines and industries (the longest of which was the Santa Fe's 43-mile Lee Ranch Coal Spur in New Mexico, which opened to traffic in 1984) there were two major new lines to coal fields opened in the 1980's. One was the Chicago and North Western's 107-mile line from Nebraska to the Powder River Basin coal fields, which opened in 1984 (see Cover photo). The other, which was the most impressive piece of new freight line construction in North America since 1961 (when the Chihuahua Pacific was completed in Mexico) was the new British Columbia Railway Tumbler Ridge line through the Rocky Mountains, which involved 80 miles of rugged mountain construction, including 3-3/4 and 5 mile tunnels, often in brutal weather (see photo 1).

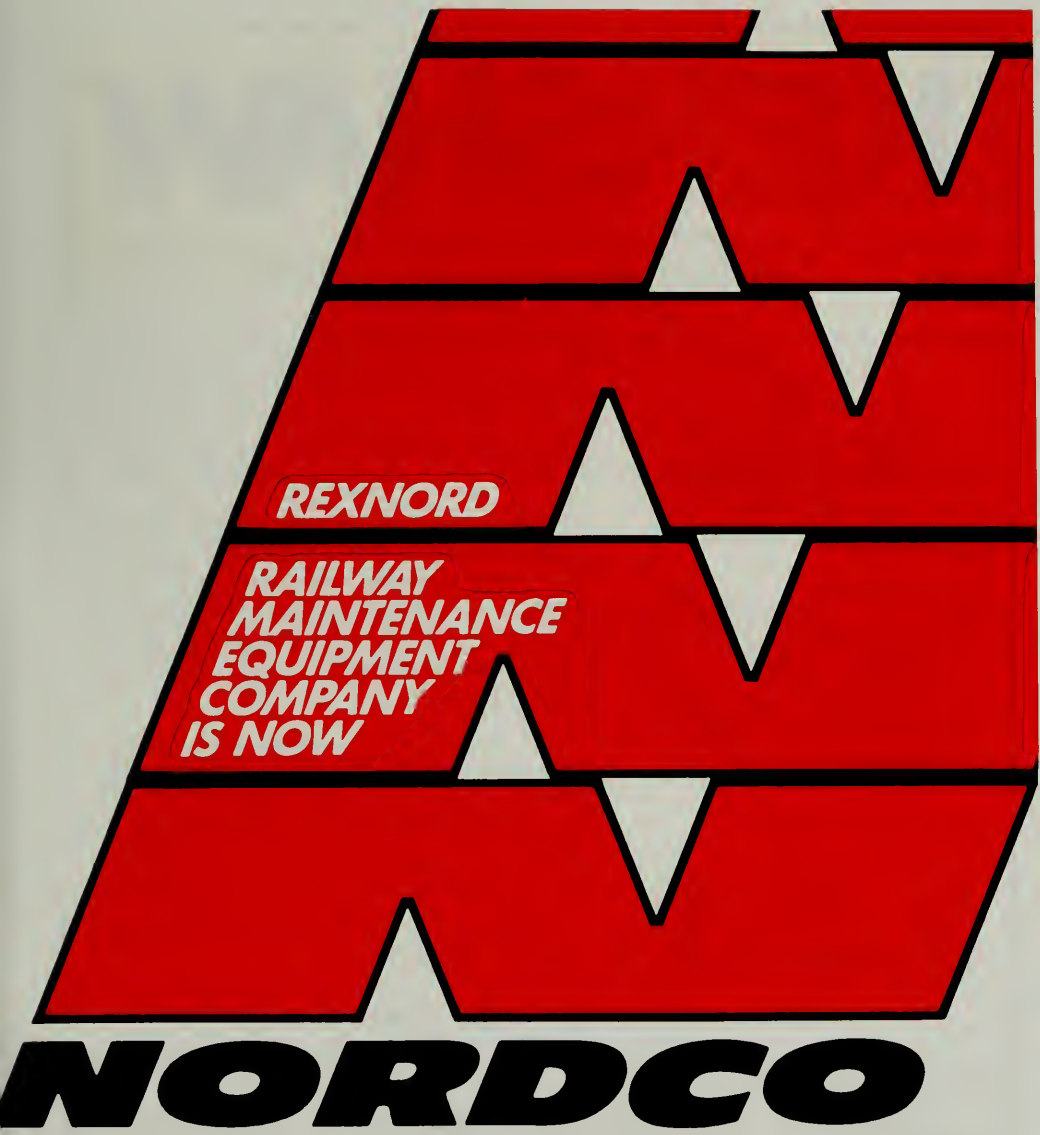
As impressive as all this new construction on freight railroads was, it is dwarfed in terms of construction expenditures by the multi-billion dollar construction of new rail transit lines. Led by new systems in San Francisco (1972), Washington, DC (1976), Edmonton (1978) and Atlanta (1979), the following completely new rail transit systems opened in the 1980's.

1981—Calgary, San Diego (See Photo 3)
1983—Baltimore
1984—Buffalo, Miami
1986—Portland, Vancouver
1987—Detroit, Sacramento, San Jose
1989—Guadalajara

In the 1980's line extensions to existing systems were constructed in Atlanta, Boston, Chicago, Edmonton, Philadelphia, Toronto, and Washington (see photo 2), as well as to several of the systems whose initial openings were in the 1980's. Extensive rail transit rehabilitation took place in Pittsburg, New York, Philadelphia, San Francisco and elsewhere.

Construction for new lines is presently underway in Atlanta, Chicago, Edmonton, Los Angeles (3 lines), Monterrey (Mexico), Vancouver, and Washington (2 lines); and many other lines are in the engineering and planning stages.

When all this is added to the upgrading of tens of thousands of miles of line to welded rail and heavier ballast sections to handle heavier loads, and when clearance improvements and battling the forces of nature (most notably in the 6-mile relocation to by-pass the Thistle, Utah slide and in the fight with the rising Great Salt Lake) are considered, the 1980's can be recorded as one proudly recalled by the railway engineering profession in North America.



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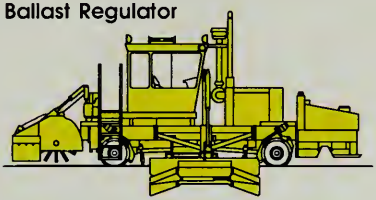

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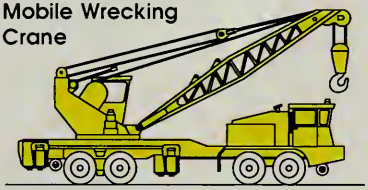
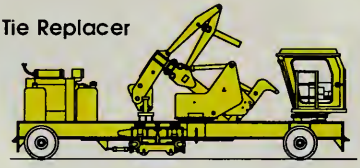


Tie Replacer



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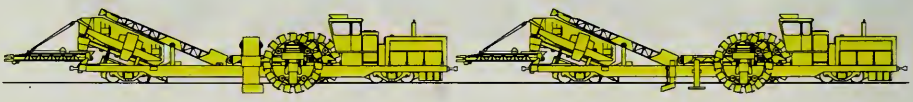

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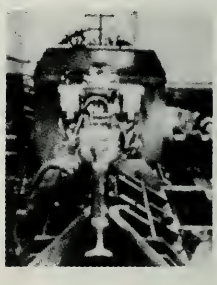

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RAIL GRINDING
THE BN EXPERIENCE

By: W. M. Glavin*

Introduction:

Rail grinding has advanced from what some have called the "traditional grinding" technique to a "profile grinding" technique over the last ten years on the Burlington Northern Railroad. At the same time, the rail grinding strategy evolved from an emphasis on correction to one of rail maintenance.

During this ten year period, our *annual* tonnage has increased from a high of approximately 50 MGT to a current high of approximately 150 MGT. From 1981 to 1987, net tons per loaded general freight car have increased 24% (from 54 tons to 67 tons), and the unit train concept has evolved to the norm for coal and grain.

Despite the adverse impacts of heavier axle loads, rail life (measured in total accumulated million gross tons) has increased approximately 25% on tangent track and from 25% to over 100% in curves.

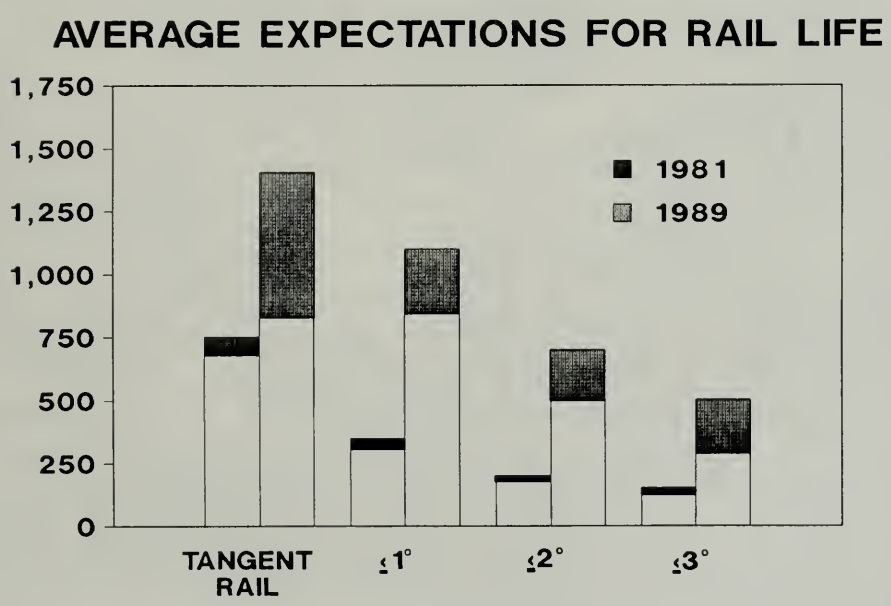

Exhibit 1.

Average expectations for rail life in 1981 were:

Tangent:		650-750 MGT	
Curves:	≤ 1°	300-350 MGT	(Standard Carbon)
	≤ 2°	175-200 MGT	(Curvemaster/Japanese Head Hardened)
	≤ 3°	125-150 MGT	(Curvemaster/Japanese Head Hardened)

Current expectations for rail life are:

Tangent		800-1400 MGT	
Curves:	≤ 1°	800-1100 MGT	(Standard Carbon)
	≤ 2°	500-700 MGT	(Head Hardened/Fully Heat Treated)
	≤ 3°	300-500 MGT	(Head Hardened/Fully Heat Treated)

*System Chief Engineer, Burlington Northern Railroad

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Advancements in rail technology (head hardened and fully heat treated rail, clean steel practices), lubrication, road bed improvement (undercutting and shoulder cleaning) and rail grinding have all contributed to this increase in rail life. However, it has been the practice of rail grinding, that has allowed the full potential of these improvements and advancements to produce the rail longevity characteristics we now enjoy.

In addition, our improvements could not have been achieved without the effort of equipment manufacturers, through the advancement of their equipment technologies, and a solid, long term commitment by our own railroad's management to provide a stable rail grinding operating base. As evidence of the stable grinding base, the amount of production rail grinding work performed on our railroad has increased from approximately 1,000 pass miles in 1978 to a current annual level of approximately 16,000 pass miles.

No one will be surprised at the statement that rail replacement is, in general, determined by two criteria; wear and both internal and surface fatigue. The primary purpose of rail grinding, is simply, to increase the service life of the rail through the reduction of rail wear and rail defects.

The modes of rail wear which offer the opportunity to be corrected by grinding are:
head wear due to plastic flow,
corrugations and spalling
side wear in high rails of curves.

The modes of rail fatigue which offer the opportunity to be reduced, delayed or eliminated by grinding are primarily:
gauge corner head check, shell, detail fractures, transverse defects,
center cracking and spall
weld geometry inconsistencies, and finally
engine burns.

The rail grinding technique that is used for minimizing these modes is not mutually exclusive and the grinding program established should be based on which parameter is currently determining the ultimate rail life.

On the Burlington Northern we have seen a progression of rail problems and solutions and have adapted our approach to meet the increasing demands of higher tonnage and higher axle loads.

— Initially the primary focus was on corrugations—now controlled;
— The advent of rail lubrication in curves dramatically increased the problems associated with gauge corner fatigue evidenced in checking, spall, shell, detail fractures and transverse defects, resulting in their development at an increasing rate—now controlled;
— Gauge face wear has been dramatically reduced primarily due to the use of head hardened rail; however,
— High hardness of the rail has created our most serious current challenge, which is the center cracking and spalling, located primarily in the center of the low rail on curves in the heat treated rails.

As previously stated, rail grinding has advanced from what some have called the "traditional grinding" technique to a "profile grinding" technique over the last ten years. The rail grinding strategy evolved from an emphasis on correction to one of rail maintenance.

Traditional rail grinding was instituted on portions of the railroads that now compose the Burlington Northern in the late 1950's. Its use was primarily for the maintenance of the rail head with regard to the elimination of corrugations on the low rail of curves, engine burns, and batter at the rail ends.

Efforts through the late 1970's and early '80's concentrated on the center portion of the rail head with virtually no work on the extreme gauge and field corners of the rail. The primary concern was in

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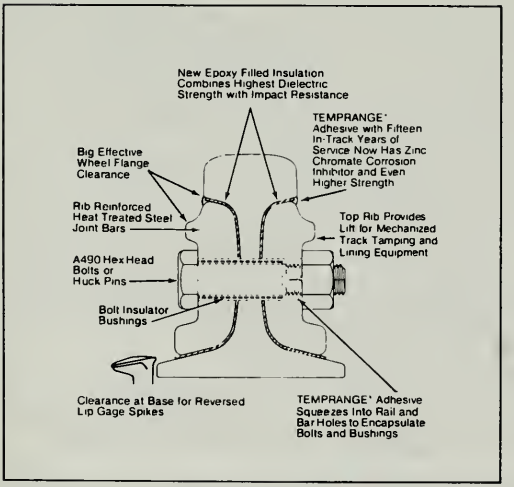
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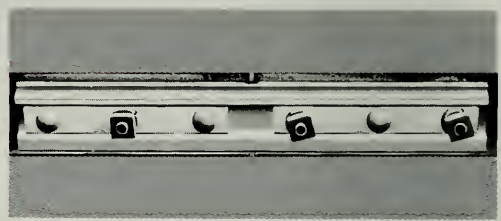

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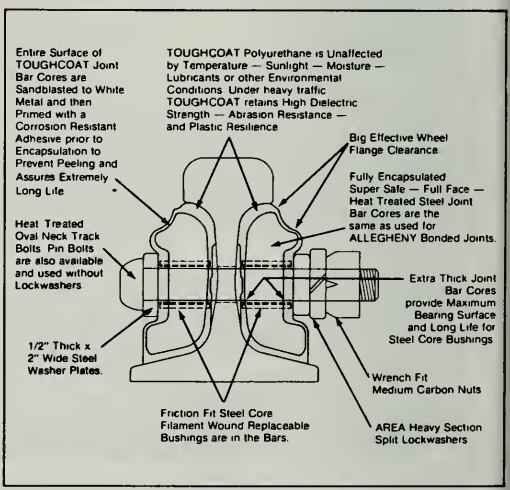
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the corrugation or surface defect reduction or removal. The resulting pattern on the rail head left approximately 1/8" of unground rail on both the field and the gauge edges of the rail head on 132# rail while on 115# rail all except approximately 1/16" on the extreme gauges and field edges was addressed.

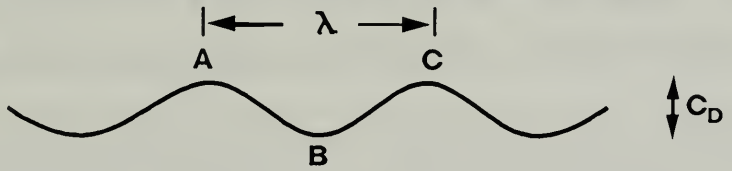
The predominant equipment in use on our property in the late 1970's was the pull-type, 96 stone machine. This machine was equipped with 96 grinding motors at 7.5 horsepower per motor, producing a total of 720 available grinding horsepower. Metal removal with this equipment resulted in approximately 5 thousandths of an inch from the rail head with a relatively equal metal removal distribution pattern.

We observed, that, in a large number of situations, damage to the rail head both exceeded the ability of the equipment to perform the required repair and our budget. Correction of corrugations became, for all practical purposes, ineffective once corrugations were longer than about 10 inches (or the diameter of the grinding wheel). In many instances, due to the severity of damage to the rail head, rail grinding of curves was used only to remove slow orders or for postponement of emergency relays, allowing the rail to be left in track until a relay could be scheduled the following spring or summer.

It was in the late 1970's that rail grinding equipment began to undergo dramatic improvements. These improvements allowed us to begin real corrective work on the rail head. The two most dramatic changes were: the increase in grinding horsepower to 1440 or 1760 (96 stones @ 15 hp per motor or 88 stones @ 20 hp per motor, respectively) and the development of corrugation control system that retarded stone drop into corrugation valleys while concentrating efforts on peaks. This new equipment allowed us to better cope with and repair more of the damaged rail surfaces.

In many cases, there was an actual increase in the initial number of passes ground per curve because of this new found ability to repair. The result, however, was that the ground rail was now being left in service for a number of years.

CORRUGATION REMOVAL

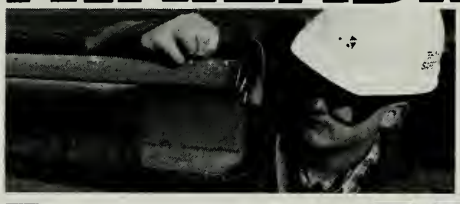


Grinding Passes	Metal Removal A	B	C	Wave Length λ	Corrugation Depth (C_D)
INITIAL	-	-	-	14"	.050
1st Pass	.015	0	.015	19"	.035
2nd Pass	.015	.005	.015	24"	.025
3rd Pass	.015	.010	.015	30"	.020
4th Pass	.015	.013	.015	36"	.018
TOTAL	.060	.028	.060	22"	.032

Exhibit 2.

PRICE

AVAILABILITY



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It is important to understand that even this new equipment had and continues to have its limitations. Since the equipment only retards stone drop, the effectiveness on corrugation removal is entirely dependent on the corrugation's length and depth. Metal removal at the "corrugation valley" is proportional to the length of the corrugation with control ending at approximately 36 inches. That is, the longer the length, the greater the metal removal in the valley. Each pass over the rail extends the corrugation length, there by reducing the effectiveness of the corrugation control mechanism.

For example:

A corrugation or surface defect exists at a depth of 0.050 of an inch over a length (or wavelength) of 14 inches.

In the first pass over this area, grinding occurs at the peaks of the corrugation with approximately 0.015" metal removal, no metal removal occurs at the corrugation valley and the resulting condition has reduced the corrugation depth to 0.035"; however, at the same time, the length is extended to 19 inches.

A second pass is then applied. Metal removal is again 0.015 of an inch at the corrugation peaks, but due to the increased wavelength, there is now approximately .005 of an inch metal removal at the corrugation valley. The resulting condition is now measured with a corrugation depth of 0.025" and a wavelength of 24 inches. Metal removal of 0.015", effective corrugation reduction of 0.010 of an inch.

Application of the third pass again results in metal removal of 0.015 of an inch at the peaks and now results in 0.010 of an inch from the valley reducing the corrugation depth to .020 of an inch, extending the corrugation wavelength to 30", effective corrugation reduction of 0.005 of an inch.

Application of a fourth pass reduces the corrugation valley depth to approximately 0.018 extending the length to 36 inches. Further passes now will provide only a uniform metal removal of 0.015 of an inch from both the corrugation valleys and peaks, effective corrugation reduction of 0.002 of an inch.

Four passes over the rail head removed a total of 0.060 of an inch from the total rail height, and have significantly reduced the corrugation depth, from .050 to .018. However, the corrugation has not been eliminated.

Due to this type of relationship between corrugation length and depth and the fact that once corrugations grow beyond a certain point they cannot be totally eliminated by today's equipment, it was apparent that effective corrugation and related surface defect control could only evolve if the equipment could be cycled on a sufficiently frequent basis. The cycle must be based on equpment limitations and corrugation growth rates, as related to time and tonnage.

Corrugation is currently being controlled to the point where unless the equipment does not complete grinding on cycle, it is no longer considered a problem.

As work progressed in reagrd to corrugation control, information from Australia on rail grinding efforts became available. Labeled "profile grinding" since the finished rail shape was now given a higher priority in the rail grinding effort, in addition to the elimination of surface defects, the next developmental step in rail grinding had evolved.

The stated goals for Australian profile grinding were;

1. Reduce the rate of side wear on high rails of curves,
2. Reduce the irregularities on the running surface of the rails, and minimally
3. Reduce the head checking problems on the gauge corner of high rails.[1]

Unique to their approach in grinding was the design of the rail profile to produce a reduction in the gauge face wear rate. This was accomplished by using an asymmetrical shape which increased the rolling radius differential of the wheels on curves and produced excellent results.

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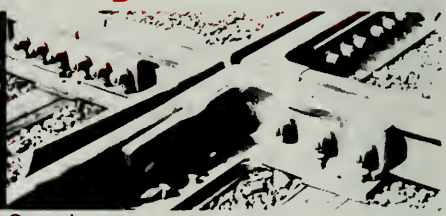
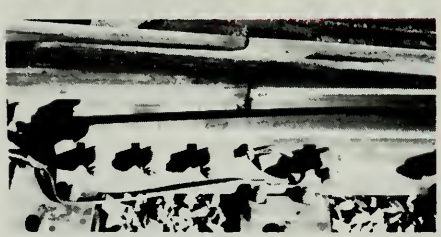


Crossings



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Attempted application of this particular profile design to North American heavy haul use did not appear practical primarily due to the false flange situation on our wheels. Modifications to the original concept were made based on the primary problem(s) faced by individual railroads.

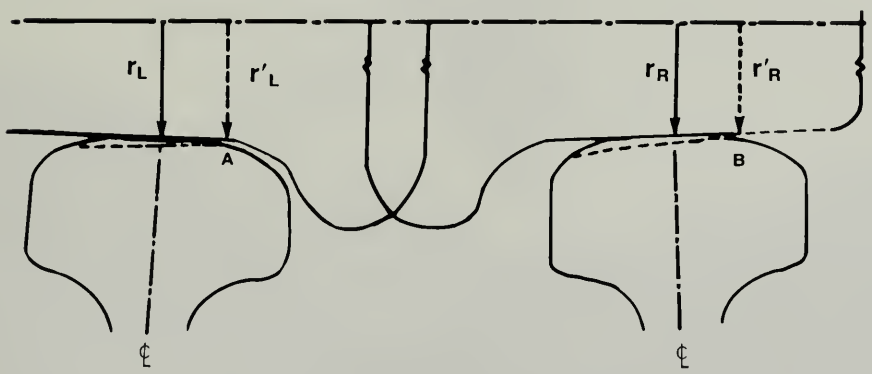
An analysis of rail conditions was performed by the CIGGT and the Canadian Pacific Railroad, priorities were set to accomplish the following results.

- Reduction of corrugation which occurs mainly in the low rail;
- Reduction of gauge corner shelling which occurs mainly in the high rail;
- Reduction of gauge face head wear which also occurs mainly on the high rail.

Lampson stated "that while it is possible to design a ground-rail profile to individually accomplish any of the above objectives, it is not always possible to achieve all the objectives using a single ground rail profile due to their conflicting requirements. In fact, the conflicting requirements are such that if maximum effort were placed on controlling shelling defects, lateral wheel forces would increase."[3]

In the same manner, if the maximum effort were placed on the reduction of gauge face wear, high on the Australian's priority list, corrugations on the low rail and associated gauge corner defects on the high rail would increase.

Profile grinding, in its most simple case, is the establishment and/or maintenance of a given rail head shape. It is important to understand that profile grinding is predicated on a relatively clean rail head. The establishment of a given rail shape without addressing existing rail head defects, in our opinion, is only completing half the job.



SYMMETRICAL PROFILE CONTACT BAND CENTERED @ r

ASYMMETRICAL PROFILE CONTACT BAND CENTERED @ r′

Exhibit 3.

The rail head is addressed in three areas or sections with basically the following concerns and intentions:

Gauge Section:

Relief or a reduction or elimination of contact is performed to prevent and reduce the development of head cracks, spalling, transverse defects and detail fractures;

In regard to the Field Section

Relief or an elimination of contact in this area is performed to prevent or reduce false flange contact and the development of corrugations;

In regard to the Center Bearing Section:

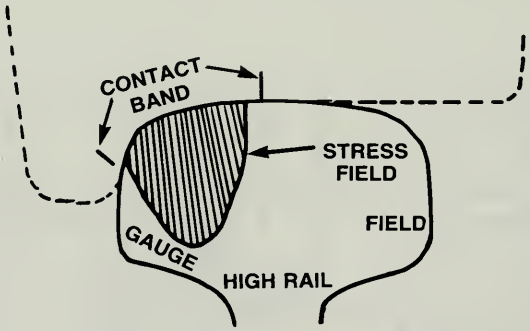
As wide an area as is practical is left in the center of the rail head for the wheel to run on. Whether the resulting profile or shape is symmetrical or asymmetrical is determined by the position or location of this center bearing area.

A symmetrical rail profile is one that has this center bearing area located at the rail head center line.

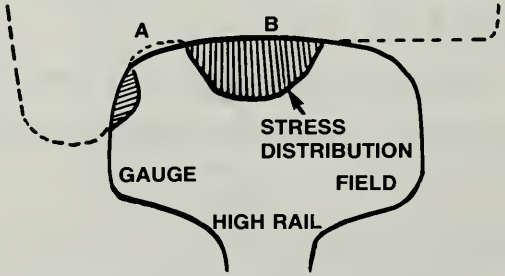
An asymmetrical rail profile is one that has this center bearing area located off the rail head center line.[3]

As you are all aware, new 132# RE rail is placed in service, the wheel/rail contact area becomes readily apparent. This contact area is generally shifted toward the gauge side of the center of the rail, especially on the high side of a curve.

Due to the single point loading of the gauge corner, the rail is overstressed, exceeding its maximum strength causing small cracks to initiate-head checks. Head checks initiate as early as 5 MGT on a 6 degree curve on head hardened rail. If no action is taken, the head checks will continue to grow in length and depth, ultimately resulting in shell or failure-detailed fracture.



WHEEL/RAIL CONTACT AND STRESS DISTRIBUTION BEFORE GRINDING



WHEEL/RAIL CONTACT AND STRESS DISTRIBUTION AFTER GRINDING.

Exhibit 4.

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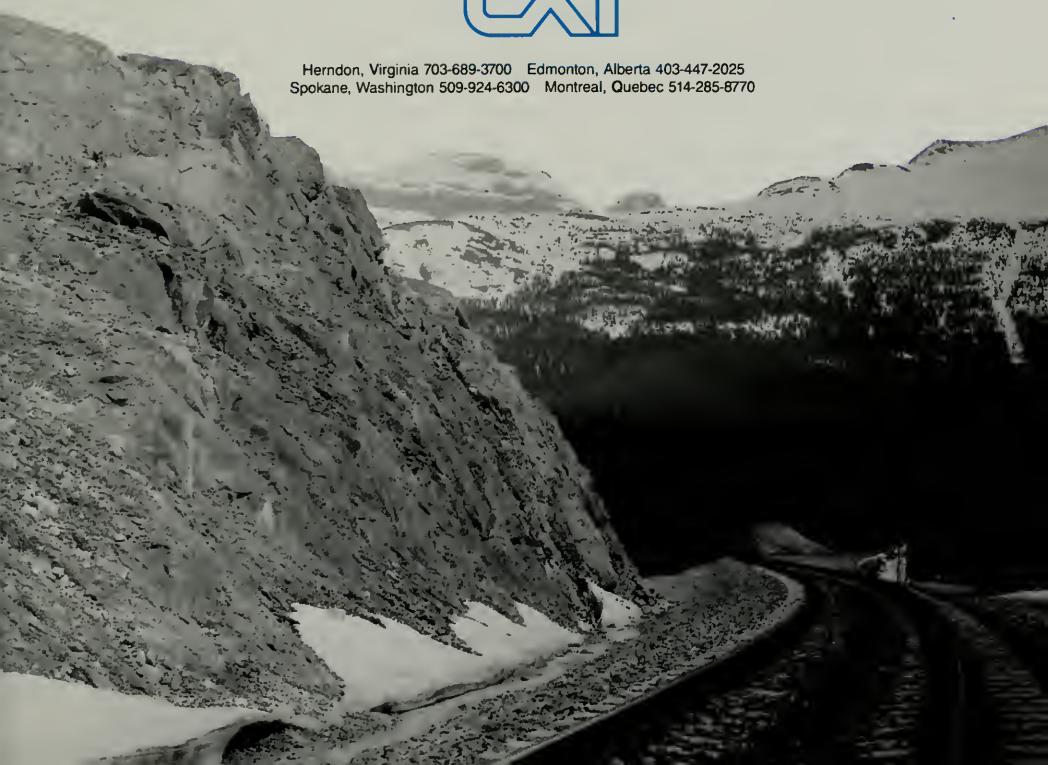

The recommended grinding action is to schedule new rail to be ground as soon as possible, removing approximately .040 of an inch from the gauge corner with a resulting radius of approximately 2 to 2.5 inches, this transfers the load to a wider area on top of the ball of the rail. A secondary benefit achieved with this action is with regard the development to transverse defects. Transverse defects are thought to initiate from inclusions in the rail. Their initiation and growth is also dependent on the stress imposed on the rail. Dr. Steele's hypothesis is that rail grinding provides an artificial wear mechanism that changes the location of the primary stress concentration, with sufficient frequency, to retard their initiation and growth.

Is it necessary or even possible to remove all the head checking?

If the rail is ground soon enough, which will depend on tonnage and location-yes. However, if the defects have progressed too far, concentration should be on removing them from the center of the rail head through the 3/4 inch gauge point, and sufficiently relieving the gauge corner. The angle between the mid-gauge transition radius and the center ball radius must be fairly shallow and smooth since the rail is in a weakened state and a radical break down could occur. Severely gauge-face worn rail must also be addressed in a very careful manner.

Is all contact on the gauge corner bad?

No. Once relieved, the rail will wear and again fully conform to the shape of the wheels. The contact band will broaden toward the gauge corner of the high rail, improve the curving characteristics of the wheels. The important factor, is to return with sufficient frequency to prevent the development of the gauge corner defects or to be able to eliminate them with one pass of the rail grinder. Have we been successful in reducing the number of detail fractures? Significant progress has been made, however, we have a long way to go.

The major problem experienced on the field side of the rail head is false flange contact. This situation is at its worst when the following conditions exist:

- worn high rail in a curve
- wide gauge exits

The relative effect on the rails, both the low and high, is compounded.

As the wheel wears the shape changes, forming a reverse curvature or false flange. The point of reverse curvature is very small, and when it contacts the top surface of the rail the stress is extremely large, resulting in the development of corrugations, center ball cracking and shelling. Wide gauge compounds the problem by allowing the false flange to travel further toward the gauge side of the rail head. This situation causes additional problems because, as it climbs up on the surface of the low rail the rolling radius of the wheel increases, driving the flange of its opposite wheel into the gauge face of the opposite rail.

The proper grinding technique is to remove a sufficient amount of metal from the field corner to eliminate or reduce contact with the false flange. This is true for rail on tangent track, as well as on the low rail of the curve. The field side of the high rail of the curve is also ground, basically a dressing action to maintain shape by removing any rail steel flow. The resulting non-contact area should also be a relatively shallow angle with a smooth radius at the transition point from center ball radius to field side transition radius.

In the center bearing area, the following conditions are desirable:

- The elimination or significant reduction of contact on the gauge corner, and,
- The elimination of contact on the field side of the rail.

The extent to which these two conditions can be achieved must now be governed by the level of stress that can be accepted in the center portion of the rail head and what its shape should be.

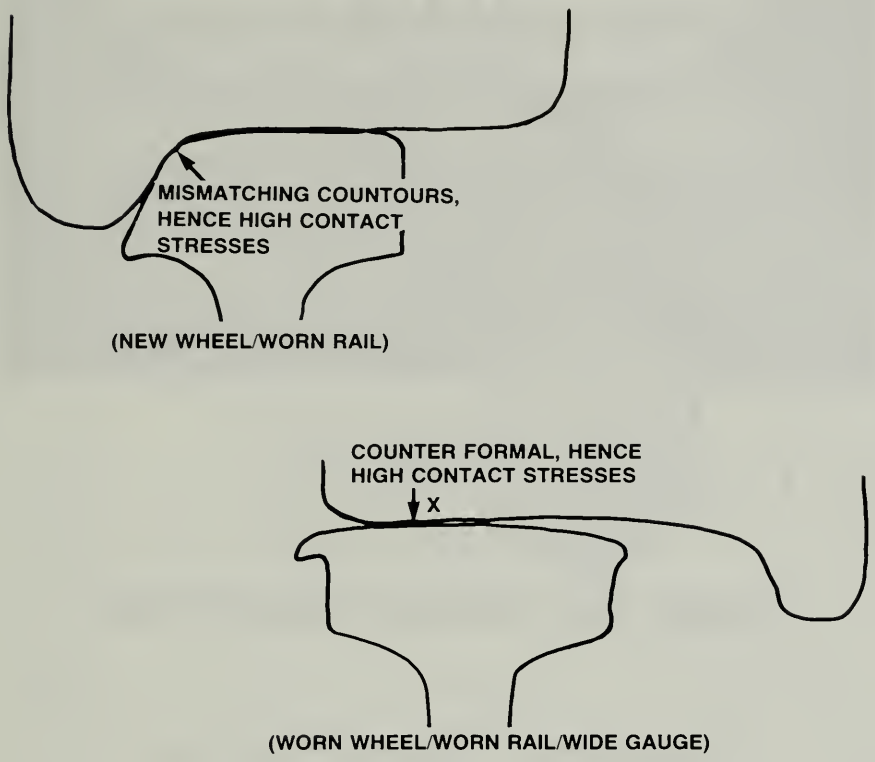

Exhibit 5.

The bearing portion of the rail head, must adequately support both the new wheel profile and the worn wheel profile, therefore some degree of curvature is required. A bearing area that is too narrow will result in overstressing of the rail head increasing its rate of flow, cracking, and/or spall.

While flow on the rail head is normal and not generally considered a problem, in most cases, small amounts of flow can be easily removed at the next grinding application. However, too narrow a contact band combined with too sharp a transition from center to the gauge or field side can result in a radical breakdown of the rail surface, especially if the rail head has deep head check in that bearing area.

Our experience has proven to us, that a contact band of 1-1/2" to 1-3/4" (with a preference toward the 1-3/4") at an 8 to 9" radius is appropriate. Contact bands in the 1" to 1-1/8" range do not provide sufficient support and dramatically increase the initiation and regrowth of center cracking. At our current cycling of equipment, contact bands in the 2 inch range do not provide sufficient relief in the gauge area to eliminate or maintain gauge corner defects, nor do they provide sufficient relief of false flange contact.

The final item left to be determined is the location of this wheel/rail contact band.

We are currently of the opinion, that the best location for the contact band, as viewed after grinding has been completed, is symmetrical, centered directly over the head of the rail (1/8 to gauge, -0 to field).

- Shifting the band to the field side of the low rail in curves, places the contact area in jeopardy with regard to the false flange of the wheel.

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- Shifting the contact band toward the gauge corner of the high rail increases the contact stresses in that area and thereby reduces its longevity. In any event, this contact band will broaden and shift toward the gauge corner as normal wear progresses between grinding cycles.

Our resulting rail head shape has a wheel contact band 1.5 inches wide, centered directly over the centerline of the rail, with an 8 to 10 inch radius.

The gauge and field corners are sufficiently reduced such that no contact exists past the point 3/4 of an inch from the centerline of the rail, however, care must be exercised to ensure that excess metal is not removed and that the transition point from the contact band to the transition area is smooth.

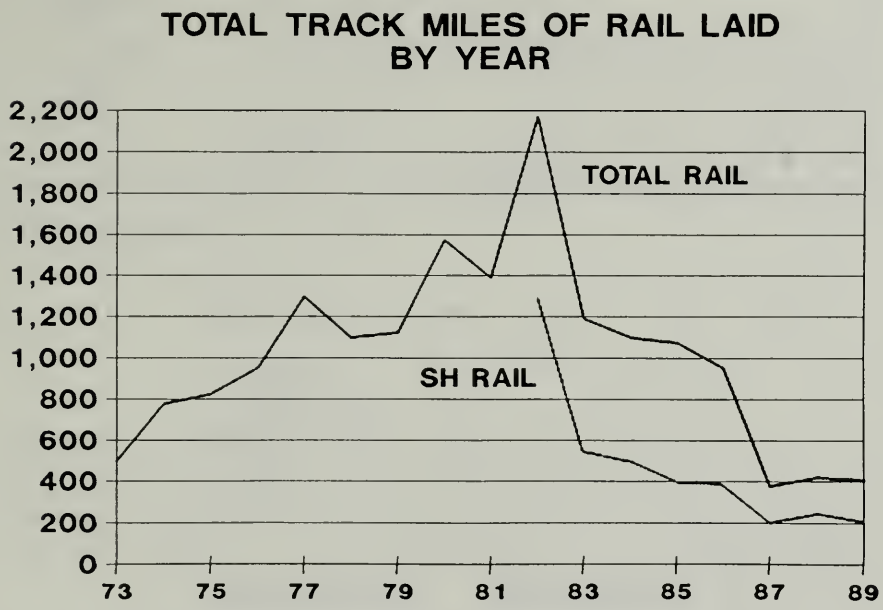

Exhibit 6.

Results

Our use of this profile grinding technique really began in 1984 with the strategy maintenance grinding. This approach to grinding rail has resulted in a reduction of our annual rail relay requirements. While it is not the only factor, rail grinding can certainly claim the lion's share of the reduction. Exhibit 6 details rail relays, both new and secondhand, since 1973. In the early 1980's, average *annual* rail relays were projected at approximately 2000 miles. Current projections are relatively level, in the 500 to 600 mile range. A large percentage of rail relayed between 1977 and 1984 has already exceeded the 750 MGT level, once thought to be the maximum service life of rail. Some of this rail has exceeded 1.2 billion gross tons.

TOTAL TRACK MILES OF NEW RAIL LAID ON CURVES ALLIANCE AND NEBRASKA DIVISIONS BY YEAR

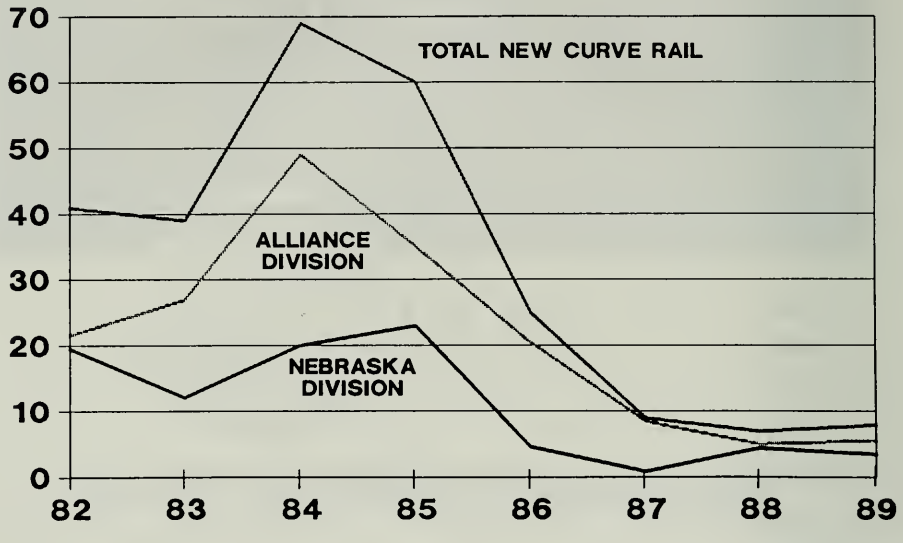

Exhibit 7.

The effect on curve relays, in a more capsulated view, is presented in Exhibit 7. It depicts the miles of curve relay performed on our Alliance and Nebraska Divisions. These Divisions handle our primary coal corridor from Gillette, Wyoming to Kansas City, Missouri. Again noting that 1984 was the year we really began our aggressive grinding approach, I think you will all agree, the change in relay requirements has been dramatic. One additional fact that should be noted is that the average annual tonnage on this line from 1982 through 1988 is over 100 million gross tons.

Exhibit 8 details an excellent example of the type of return one can generate through the effective use of rail grinders. Curve 77, between Bridger Junction and Donkey Creek, Wyoming is in the heart of the coal country. This curve was originally laid new in 1979. As part of our "NEW" Gillette-Orin line, traffic did not begin running over this rail until late 1979. No rail grinding work was performed on the curve through 1981. The curve was recommended for relay in 1982, at what would have been an accumulated gross tonnage of 150 million gross tons. In April of 1982 the rail was ground, 2 passes on the high rail and 3 passes on the low rail, this resulted in a rescheduling of the relay for 1983. This rail was again ground in April and October of 1983 with a total of 6 passes on the high rail and 9 passes on the low rail, again resulting in a postponement of the relay to 1984. In 1984, the rail was finally relayed with new 132# Japanese Head Hardened rail at an accumulated gross tonnage of 300 million. Two additional years of service were achieved and the accumulated tonnage was doubled, at a cost of approximately $3,600. The net present value analysis shows a savings of approximately $7,700 with a Return on Investment of 224%. While this is a well-above average performance, it is not all that unusual and in this case, it appears to be money well spent.

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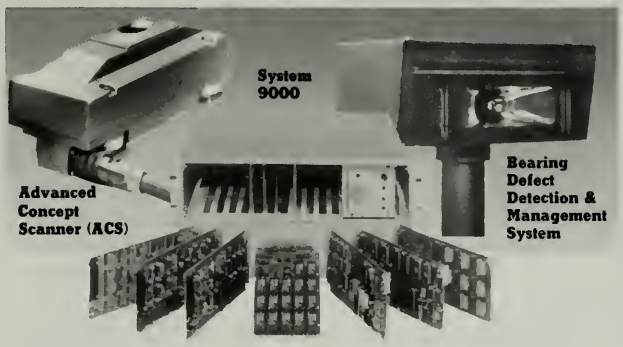

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CURVE GRINDING ECONOMICS

Year	Grinding Passes	Cost/ Pass Mile	Total Cost
1982	3	$1003.87	$1,024
1983	9	$839.23	$2,568
			$3,592

Rail relay postponed from 1982 to 1984

Cost of Relay = $205,000/mile x 0.34 mile = $69,700

Life of rail extended from three to five years.

Annual ownership cost of rail reduced from $26,075 to $16,547

saving $9,528 per year

Exhibit 8.

CURVE 77 FOLLOW-UP

Denver Division
Curve 77 3° 0.34 track miles
Rail: 132#RE Standard Carbon CF&I
Laid New in 1979

1982 Recommended for relay @Accumulated tonnage of 150 MGT
1984 Actually Relayed @Accumulated tonnage of 300 MGT
Rail: 132#RE Head Hardened
1989 Recommended for relay @Accumulated tonnage of 600 MGT

Exhibit 9.

Exhibit 9, presents a brief summary of the history of Curve 77 to date. As previously mentioned, at the time of the initial proposed relay, the 1977 standard carbon rail in curve 77 had accumulated approximately 150 million gross tons. When actually relayed, the accumulated gross tonnage was 300 million. The 1984 rail that was relayed, is currently in track and has accumulated approximately 540 million gross tons and is scheduled for relay this year when the accumulated tonnage will approach 600 million gross tons again doubling the life of the rail.

We have proven to ourselves that grinding rail can increase the service life of rail in a cost effective manner. The future in rail grinding for us will be relatively a five-fold approach:

1. Initiate an as-rolled rail profile close to the ground rail profile, thus reducing the amount of material that must be removed on the initial grinding pass. We expect to have this rail in track this year;
2. Intelligent and accurate rail measurement systems that can feed intelligent, accurate and consistent rail grinding equipment with preprogrammed profiles that will increase uniformity and minimize metal removal;
3. Increased effort in the analysis, location, and correction of the track structure inconsistencies that increase the rail head degradation and the required rail grinding effort. Inconsistencies such as low joints, sags, over-elevation in curves, etc.;
4. Improved identification and definition of the maintenance grinding cycle that will allow frequent applications at higher speeds, again reducing the rate of metal removal from the rail head on a per MGT level; and
5. Modifying our approach for the 125 ton car.

Rail grinding has advanced from what some have called the "traditional grinding" technique to a "profile grinding" technique over the last ten years on the Burlington Northern Railroad. At the same time, the rail grinding strategy evolved from an emphasis on correction to one of rail maintenance. During this ten year period, our *annual* tonnage has increased from a high of approximately 50 MGT to a current high of approximately 150 MGT, net tons per loaded general freight car has increased appreciably, the unit train concept has evolved to the norm for coal and grain complete with 100 ton cars and we are now at the initiation point of 125 ton car usage. Despite the adverse impacts of heavier axle loads, rail life (measured in total accumulated million gross tons) has increased approximately 25% on tangent track and from 25% to over 100% in curves. Advancements in rail technology, lubrication, road bed improvement, and rail grinding have all contributed to this increase in rail life.

However, in our opinion, it has been the practice of rail grinding with the profile grinding technique and the maintenance grinding strategy, that has allowed the full potential of these improvements and advancements to produce the rail longevity characteristics we now enjoy. Finally in closing I like to state that, our improvements could not have been achieved without the effort of equipment manufacturers, through the advancement of their equipment technologies, and a solid, long term commitment by our own railroad's upper management to provide a stable rail grinding operating base.

References;

[1] "Determine the economic timing for grinding and renewal of rail"
M.D. Roney, S.T. Lampson, M.B. Baggott

[2] "Development of Rail Profile Grinding at Hamersly Iron"
B. H. Longson, S.T. Lampson

[3] "Rail Profile Grinding" CIGGT Report No. 82-7 November 1982
S. T. Lampson

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Environmental Control Through Engineering Design and Operations Control

By: M. J. Boyer, P.E.*

1. INTRODUCTION

Having watched and worked with the railroad industry in its dealings with environmental issues over the past 20 years, I have observed a developing maturity and an increasingly responsible approach to environmental matters. The history of this is summarized in this series of anonymous quotations and paraphrasings from the railroad industry.

1968—"What environmental law?"
1970—"Does this really apply to us? It will all blow over."
1974—"Maybe we ought to put in a gravity separator at the servicing area—how much fuel are we spilling, anyway?"
1980—"We have spent a good deal of money on environmental control. Now it surely will all blow over."
1984—"Fuel and lube oil lost is money lost. We subsequently cannot afford the cost of pollution, and we will not tolerate it at our yards."

Today the last statement, more than ever reflects the very responsible and, I believe, very practical attitude that the railroad industry has adopted towards environmental compliance.

In addition to being uneconomical, pollution is no longer acceptable. The public will not tolerate it, your company management will not tolerate it and your stockholders will not tolerate it—and, of course,the state and Federal governments, reflecting this attitude, will not tolerate it either. If it previously were unclear whether this would all blow over, it now should be evident that, if environmental controls have continued to tighten during the Administration of the past eight years, this trend can be expected to continue.

Where is all this headed? The main theme of environmental challenges of the next decade is a recognition that they must be dealt with by considering operations modifications and engineering controls to minimize environmental compliance costs.

One of my earliest experiences in handling environmental problems for the railroad industry was for a small short line in Georgia (since absorbed by one of the larger present-day roads). During the course of that work the elderly CMO, now long since retired, related to me that this was not the first oil/water separator to be placed at this yard. The previous wooden unit was installed in the late 1940's when diesel locomotives first appeared on the scene.

The objective was an environmental need—to prevent reoccuring incidents of the creek downstream from the yard catching on fire. But the project had a side benefit. They recovered sufficient diesel fuel to meet their needs for oiling roads and thereby saved enough money in deferred road oil costs to easily pay for the separator. He added that it never occurred to them at the time to put the recovered oil back into fuel storage or use as boiler fuel.

A simple story but it embodies an important issue—this railroad was able to solve an environmental problem using—at the time—innovative engineering and turned the entire matter into a profit! Maybe they should not have been spilling the fuel to begin with, but it was 1949! More importantly, it embodies an attitude and approach to environmental and problem-solving which today's railroads must implement and carryover throughout the next decade and beyond if they are to prevent environmental compliance costs from becoming the tail wagging the dog.

Whenever possible the modern railroad must use its engineering and operations resources in a pro-active manner in order to solve environmental issues before they become problems. This process is underway on most railroads. A simple example is the use of automatic fuel shutoff nozzles in order to prevent spillage. We, as railroad engineers must be on the forefront of resolving environmental problems, and more importantly, preventing the problems from becoming problems in the first place. This is not unlike preventive maintenance.

*President, Applied Engineering and Science, Inc.

This call for pro-active environmental control is not only directed at that most miss-understood individual, the railroad environmental engineer, but also and perhaps more importantly at the remainder of the railroad engineering function. This includes maintenance-of-way, facilities engineering, B & B, mechanical, research and test and others. The railroad environmental engineer has typically been the problem solver, usually responding to an existing situation. The remainder of the engineering function is really more in a position to identify and prevent problems from occurring in the first place. However, this is a responsibility which must be implemented as a coordinated effort with management backing and funding if it is to succeed.

For this to happen the organization must recognize that environmental decisions within industry are really business decisions. This is not to suggest a lack of genuine concern for environmental matters, but rather, that engineering expenditures will ultimately be judged to a large degree on a benefit/cost basis. In the past many decisions have been made neglecting environmental considerations and were not good business decisions as a result.

II. FUTURE NEEDS

I see the future of railroad environmental problems in four major areas.

1. Hazardous waste/groundwater contamination problems, including leased properties.
2. Contaminated stormwater runoff problems not previously addressed from shop and yard areas.
3. Environmental response to derailment incidents.
4. Increased wastewater handling costs due to unconventional/toxic pollutants handling requirements as well as increased operator attention to meet more consistent performance demands.

This is not meant to be an all-encompassing list. Certain roads may have unique problems not addressed—but every railroad has or will encounter problems relative to the four items prioritized above.

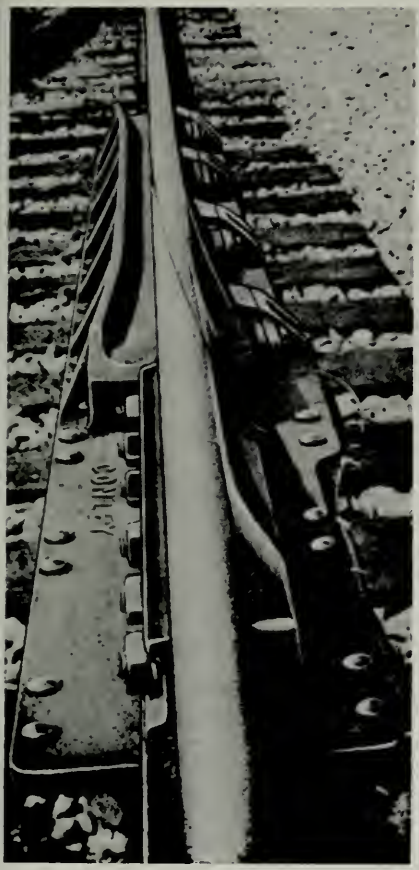
A. Hazardous Waste/Groundwater Contamination

As you are all probably aware the railroad industry will direct a major portion of its environmental effort and budget in the next decade towards resolving problems created in previous years. These are primarily in the form of hazardous waste/groundwater contamination problems due to past activities at rail shop and servicing areas, along with inherited leased property problems. This is unfortunate, of course, because it frustrates the need to deal with issues before they become problems. It does emphasize that the railroads have, to some extent, been their own worst enemy in the past. To quote Pogo, "We have met the enemy and he is us."

I do not believe, however, that the railroad industry should be judged too harshly, as the rest of the industrial/manufacturing base in the United States is in about the same situation. However, it does provide us with the opportunity to prevent a reoccurrence, as we will surely be held accountable if we find ourselves 10 years from now facing a new set of problems which we have allowed to develop.

This presentation does not address the extreme technical complexities of this subject but contains recommendations in two areas which address a pro-active approach to preventing the next generation of problems from occurring.

1. Rail property and facilities—There is little excuse for the creation of new problems at shops, servicing areas and other properties owned or operated by the railroad. Experience has shown, however, that this is easier said than done, on a rail system which may cover several thousand miles and many states. The organizational aspects of this problem will be discussed later. However, there is a role here for every element of the engineering function. Things happen on a rail system. Well intentioned and necessary activities can create new hazardous waste/groundwater contamination problems, often without the knowledge of management or the railroad environmental engineering functions—even in a well run operation.



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All elements of the engineering functions have responsibilities:

a. You must have a modest knowledge of environmental acceptability and
b. Be aware of what is happening on the part of the system for which you are responsible, in order to prevent problems before they happen or at least identify a problem in the early stage.

This applies to maintenance-of-way, B & B, mechanical and the other engineering functions.

This is not limited to out-of-way occurrences. An example is an increasing use of contract tankwagon fueling of mainline locomotives, with no more consideration for environmental protection in many instances than the truck driver who does the unloading—and he is not even a railroad employee. The potential for short and long term environmental damage is great and all the indemnifications and third party contracts will not prevent the railroad from incurring direct and indirect expense in response to this.

The pro-active approach to environmental control must include a realistic assessment of environmental risk and associated remedial cost before such projects are undertaken.

2. Leased properties represent a very challenging and often frustrating aspect of rail system pro-active environmental control. Two areas are recommended.

   a. Railroads must maintain an ongoing audit of all leased property holder activities to insure that they conduct operations in a manner to prevent at least major environmental problems from developing which would likely be inherited by the railroad.
   b. The various elements of the railroad engineering function have a responsibility to know, within reason, what is environmentally acceptable, and to report within the railroad when something does not appear acceptable on leasee property. For example an observed continued history of spilled commodity from an adjacent property car unloading operation should be reported to the Division Superintendent. If this is a leased property it may prevent a major future problem and expense to the railroad.

B. Stormwater Runoff

The railroads, of course, have dealt with the problems of handling contaminated stormwater runoff for many years, primarily oily problems from diesel locomotive and ready track areas. This is not a completely new subject, as it is to much of the rest of the industrial world.

Over the past six to eight years the Federal and state regulatory agencies have wrestled with the problem of dealing with contaminated stormwater runoff from sources less obvious than servicing areas. The origin of these more recent developments was a lawsuit filed and won by the Environmental Defense Fund against EPA. The action contested the manner in which EPA handled the issue. The outcome resulted in amendments to the 1987 Federal Clean Water Act which essentially requires the states to develop and implement regulations for monitoring and substantially reducing contaminate levels in stormwater runoff. This will be based on local water quality criteria and related factors.

As this generally relates to industry, the states will develop and prioritize various classes of storm runoff contamination sources. However, it is now evident that industrial plant sites will be in the first class to be addressed. This will clearly include major railroad shop areas. Now in question is whether classification and other yard areas will also be included.

Final guidelines are scheduled for publication in the Fall of 1989 with monitoring and evaluation of individual outfalls occurring in 1989/91. Major capital expenditures will be required in 1991-94. Compliance will require a mixture of loss control improvements through Best Management Practices (BMP) to reduce or prevent losses contributing to the problem, in addition to facilities for collection and treatment for some amount of site stormwater runoff.

A number of sites have been required to address this issue at this time due to unique environmental settings, particularly severe runoff problems or other circumstances. Our observations indicate that shop sites and yard areas will be faced with significant challenges and major expenditures to meet the anticipated requirements of these regulations.

The key to minimizing expenditures will be the implementation of BMP to the maximum extent practical. What does this mean for railroads? In short, keep from spilling and dumping foreign materials, in particular chemicals, oils and solvents, and thereby prevent contamination from occurring. The participation of all parts of the railroad engineering function is required in order to minimize railroad cost exposure. The extent to which railroad management chooses to train and direct other resources at the problem will be a business decision.

C. Derailments

Derailments are railroad emergencies. They often require massive deployment of railroad personnel, equipment and other resources on short notice. Decisions are made in the field, often with limited information.

In the past the primary goal was to clear the right-of-way and restore service. At times this is done without regard to the costs to the railroad of environmental and other secondary effects of the restoration activities.

I want to emphasize that this does not generally relate to consideration of immediate safety for rail personnel and the general public. Railroads have a long history of making this the first priority.

However, I recall an incident on which I was part of the first response team to a derailment involving massive caustic soda spillage. The track crew busily went about restoring track as soon as possible, at the expense of ignoring (over our pleas) the fate of several thousand gallons of 50 percent caustic soda which reached a major waterway. The fish kill, water supply curtailment and related damages over the next few days cost that railroad over $1,000,000—for the right to open a stretch of track immediately that handled about two trains a week.

Now, I am happy to also report that the incident occurred over 15 years ago and I am sure that no railroad would handle it in this manner today.

Clearly the message in this story is the need to make informed, on-the-spot engineering judgments that consider not only the need to restore service as soon as possible and subsequently minimize revenue loss to the railroad, but to also consider the secondary impacts and costs to the railroad. While there are environmental considerations, the reality is that they ultimately become business decisions. Therefore, the engineer at the site must be able to assess the situation and make decisions, usually in conjuction with upper management who are remote to the derailment. The overall decisions must consider the potential revenue lost due to train traffic rerouting and delays versus the deferred environmental costs by focusing primary emphases at the proper place.

The point is not to always place environmental needs above traffic restoration, but to make good business decisions regarding the costs of each.

What can be done to minimize environmental costs to the railroad prior to a derailment incident? A good approach is to assess and prioritize various segments of the rail system in order to determine where derailments would occur in the most environmentally sensitive areas. Track and equipment inspection and maintenance can be increased in these environmentally sensitive—and expensive—corridors.

This is not to suggest that derailments are more acceptable or that environmental protection is not important in other areas. The costs of response and environmental damage is greater and it is a good business decision to be aware of this and direct resources appropriately.

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D. Waste Treatment

Impacts will be felt in two areas resulting in significant railroad cost exposure.

1. Railroads will be required to address wastewater treatment, particularly at large shops, in order to handle various non-conventional pollutants.

2. Railroads will be required to address operator attention at many facilities in order to maintain more consistent achievement of discharge limitations.

Now, is this area the bailiwick of the railroad environmental engineer or does it involve several other aspects of the railroad engineering function? If railroads wish to deal with these issues as reactions to problems rather than pro-active prevention then no one else need be involved!

1. Treatment facility upgrades/non-conventional contaminates—What are these non-conventional contaminates and where do they come from? One thing for sure, the railroad environmental engineer does not use them nor does he allow them to reach the shop sewer.

   In early 1980's I worked on a major diesel locomotive rebuild shop in the midwest which had a consistent and unusual problem with high levels of phenols. We had performed a substantial amount of facilities and environmental work for this railroad so we were asked to determine what improvements would be necessary to upgrade existing treatment facilities to remove this level of phenols. Having worked with phenolic wastes in the past it was clear that the costs to remove 5 to 10 mg/l phenols in 300,000 gallons per day of wastes was going to be staggering relative to everyone's expectation at the railroad. Also, that level of phenols does not just appear out of nowhere.

   Therefore we encouraged and convinced the railroad to let us try to first solve this at the source. It did not take much investigating to determine that locomotive exteriors were being stripped with a phenolic paint stripper. This was a common problem in commercial aircraft maintenance waste handling and we had previously dealt with it on a number of occasions. The mechanical shop forces and probably Research and Test group knew that phenolic strippers were being used but the organization structure to relate this knowledge to the problem was not in place. The railroad investigated several other methods of paint stripping and eventually selected one which solved the problem.

   This will be a reoccurring story for the railroads as regulatory agencies tighten limits and further add limitations on parameters such as boron, heavy metals, chlorinated and other "toxic" hydrocarbons (solvents). California—the nation's leader in thinking up problems for the railroads which they didn't know they had—has passed Proposition 69. This essentially requires a zero discharge of a long list of cancer causing chemicals in permitted wastewater discharges. Other states are considering similar legislation at this time.

   The solution to this simply cannot be to expand and modify existing waste treatment facilities, although some of this will undoubtedly be necessary. Rather a coordinated effort involving environmental engineering, mechanical along with Research and Test departments is needed to define these problems and ways to solve them at the source. The time to start is now, not when you receive your next draft permit.

   I also believe that a process such as this has the potential to identify "spin-off's" in a great many areas for improvement and cost reduction, many unrelated to environmental controls.

2. Operations improvements—It is only within the past few years that most railroads have installed data aquisition and management systems to reliably determine whether wastewater treatment facilities were in compliance and how often. Unfortunately this has been about equally matched by an overall setback in the number of trained operators. Many are leaving the railroads due to buyouts and other labor reductions.

A major focus of the regulatory community is monitoring and enforcement of existing permits. This will clearly result in the need for better operation. Further, the increased treatment techrology which will be necessary at some locations due to item 1, above, will require not only more operators but also better educated and better trained operators.

The current mass exodus of experienced environmental engineers and operators from the railroad community will be reviewed in retrospect as a tragedy when the full cost impact of it is realized by railroad uppper management. Unfortunately this realization will probably come in the form of noncompliance notices and Consent Orders.

III. ORGANIZATION AND IMPLEMENTATION

All of what has been said is good, but how do we make this work. A few basic concepts are outlined.

1. Most importantly, each railroad must develop and implement a realistic environmental policy which is consistent with its business policies and objectives if not, the dollars will win every time. This may, in turn, require some modification to the business policy but the goals and objectives of each must be consistent.

2. An effective organizational structure must be established within each railroad that defines responsibilities within the various engineering and operation functions. Equally important, lines of communications must be set up to insure proper participation by all parties if a pro-active environmental control program is to function.

   Is there a best organizational structure? I think not. There are no two railroads that function the same. Each organization has strengths and weaknesses and individual personalities to contend with. Clearly, however, the organizational structure will have some common components:

   *the environmental engineering function will probably play a lead role

   *major divisions of engineering and operations must be involved. This includes Maintenance-of-way, B & B, Mechanical, Research and Test and Transportation as a minimum. A clear definition of goals, objectives and responsibilities for each must be established.

   *sufficient training of these other operations and engineering functions must be instituted in order to insure their ability to perform the defined tasks

   *the organizational structure must be consistent with the remainder of the railroad organization and with the railroad business objectives.

   *the organization must have sincere backing of upper management.

   *and, of course, the inevitable law department presence will be needed

3. The lead role in this organizational structure will require a person or persons with the ability to obtain management commitment and garner an appropriate share of finite corporate resources.

Railroads will find that implementation of such a pro-active environmental control program is not at all difficult once the proper organization and communication structure is in place. Further, railroads will find this to be a very financially rewarding experience, solving problems before they develop, along with the spin-off gains to be realized.

The alternative, as pointed out in the INTRODUCTION, is the tail wagging the dog method of responding to problems after they develop, and the railroads cannot survive into the 21st century using that approach.

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DEVELOPMENT AND MANUFACTURE OF DEEP HEAD HARDENED (DHH) RAILS

By: K. Abe*, T. Miyata*, K. Sugino**, H. Yoshitake***, T. Suzuki****

1. INTRODUCTION

Railroad companies have in recent years made many breakthroughs in quest of improved efficiency in transportation by employing higher-speed trains and heavier-haul freight cars. This move has obviously created a new demand for rails which can withstand the higher-speeds and greater weights.

To meet these requirements, Nippon Steep Corp. developed a New Head Hardened (NHH) rail in 1978, followed in 1978 by the "Super Rail," aimed at preventing the welded joints from softening. These new types of rails have since been enjoying widening acceptance in the United States, Canada, Brazil and many other countries for use with heavy-haul cars.

More recently, however, the railroad industry has been faced with new challenges stemming from changes in rail operating conditions:

(1) more thorough rail lubrication aimed at improved fuel economy and reduced rail wear
(2) grinding of rail heads to remove surface damage and insure better rail-wheel matching
(3) increased use of heat-treated CWR for heavy-haul applications
(4) increase in haul per car from 100 tons to 125 tons

The Deep Head Hardened (DHH) rail in question is Nippon Steel Corp.'s latest answer for these requirements.

2. OBJECTIVES IN DEVELOPING NEW HEAT-TREATED RAILS

In developing the new market-oriented rails, we set the following objectives:

(1) Capability of selectively imparting the desired strength to rails depending on the service conditions

The use of lubrication reduces rail wear and causes fatigue layers to accumulate on the rail surface. To prevent this, periodic grinding of the fatigue layer has usually been resorted to. And alloy rails have been employed in tangent tracks as medium-strength rails.

To replace these conventional practices, it is necessary to selectively produce rails by heat treatment with a hardness suitable for their use.

(2) Having a deep hardened layer

Rail head grinding requires that the rail interior has the same wear resistance as the rail surface. This means that the hardened layer should be made as thick as possible.

(3) High weldability

Of similar importance are high weldability and high performance of the weld joints.

(4) Homogeneity throughout the length

In order to ensure a high level of rail safety under increasingly severe conditions, it is important to produce rails which are homogeneous throughout the whole length.

The off-line heat treatment process can't be applied to the manufacture of rails which meet the above-mentioned requirements without sacrificing production efficiency.

Therefore, we entered into the development of a new type of heat-treated rail using what we call the Direct Heat Treatment Process which utilizes the heat remaining in the rail directly after hot rolling.

*Quality Management Div., Yawata Works, Nippon Steel Corp.
**Yawata R & D Laboratory, Central R & D Bureau, Nippon Steel Corp.
***Shape & Stainless Steel Plate Div., Nippon Steel Corp.
****Heat Technology Laboratory, Central R & D Bureau, Nippon Steel Corp.

3. DEVELOPMENT OF DIRECT HEAT TREATMENT TECHNOLOGY

(1) Selection of coolant

A laboratory test was first conducted on the six different coolants shown in Table 1.

As a result, three candidates, salt, air, and mist were chosen and then subjected to a simulation test.

Table 1. Results on Cooling Performance of Rail Head by Various Coolants

Coolant	Temperature and concentration	Quenching conditions	Planned rail strength	
			High strength (BHN ≒ 380)	Medium strength (BHN ≒ 300)
Boiling water	98℃	Steam injection	Insufficient cooling capacity	Formation of abnormal structure
Hot water	65~80℃	Immeration	Formation of abnormal structure	Formation of abnormal structure
Oil	100~180℃ Martempering oil	Impeller agitation	Severe manufacturing conditions	Formation of abnormal structure
Salt	400~550℃	Nitrogen gas agitation	Manufacturable	High cost
Mist	Normal temperature	Water and air mixed at right angles	Unstable structure	Manufacturable
Air	Normal temperature		Insufficient cooling capacity	Manufacturable

(2) Test results of the simulation test

a. Results of test using salt (50% KNO_3, 50% $NaNO_3$)

The test results of test using salt are shown in Figure 1. With a salt bath temperature of 400°C, it is possible to obtain high-strength rails comparable to NHH rails. However, if medium-strength rails (with BHN ranging from 320 to 340) are to be produced, it is necessary to maintain the salt bath at 550°C, a temperature level at which salt will evaporate. Besides, since the immersion of rails causes the salt bath temperature to rise, a large-scale heat removal system is needed if a salt bath is employed on an industrial scale.

b. Results of test by mist cooling

The mist cooling test results are shown in Figure 2. With mist cooling, production of high-strength rails is possible, at least from the viewpoint of the cooling rate. But the cooling rate varies greatly depending on the surface quality of the rails, especially the effect of scale, as indicated in Figure 3, thereby causing localized growth of bainitic and martensitic structure (Figure 2). This makes it extremely difficult to obtain uniform structures throughout the length of a rail.

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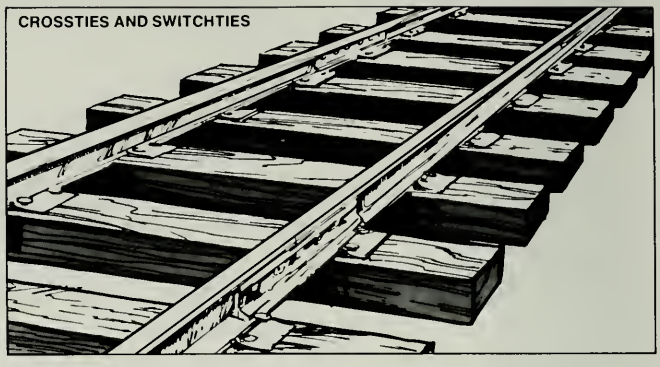

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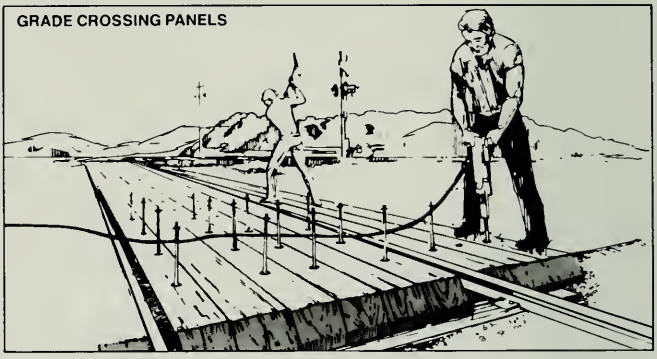

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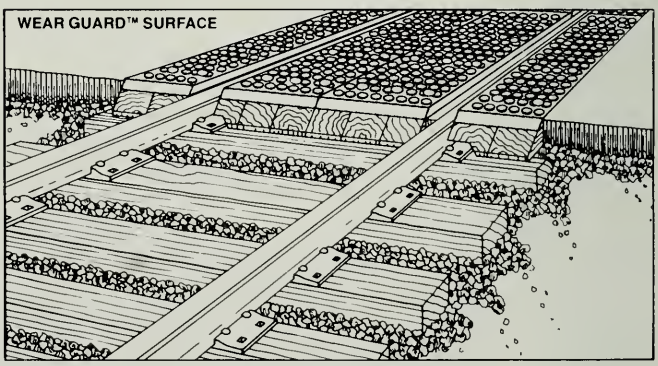

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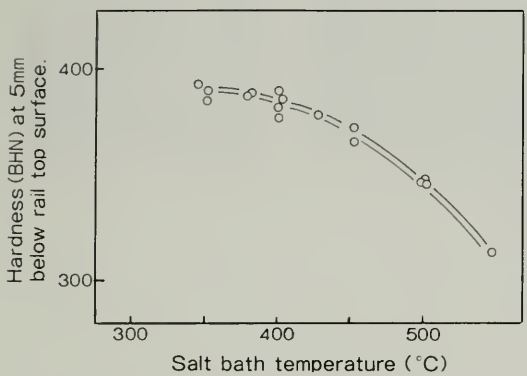
Figure 1. Hardness of Rail (Test Using Salt Bath)



〈Salt bath temperature: 400°C〉

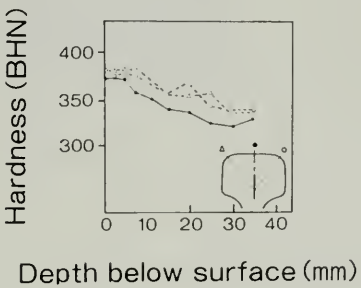


Figure 2. Hardness & Macrostructure of Rail
(Test Using Mist)

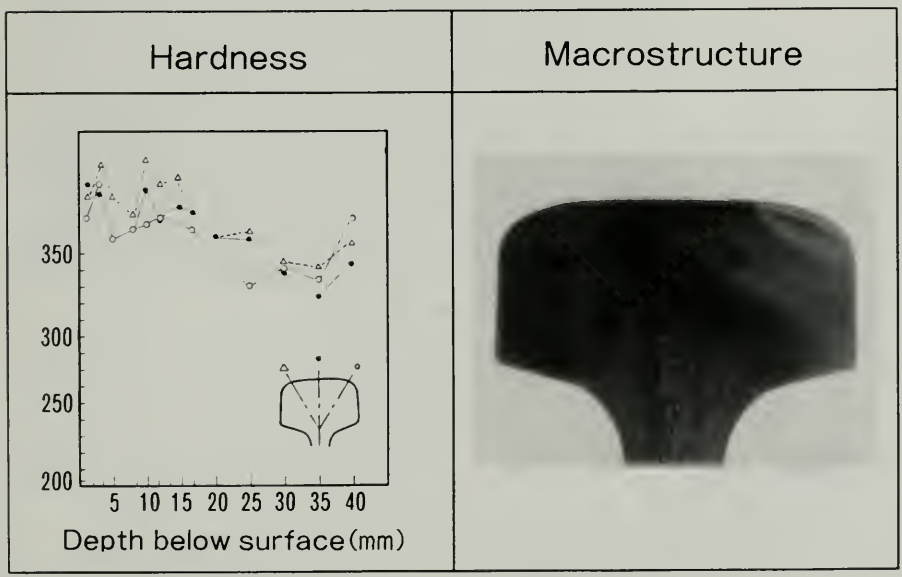
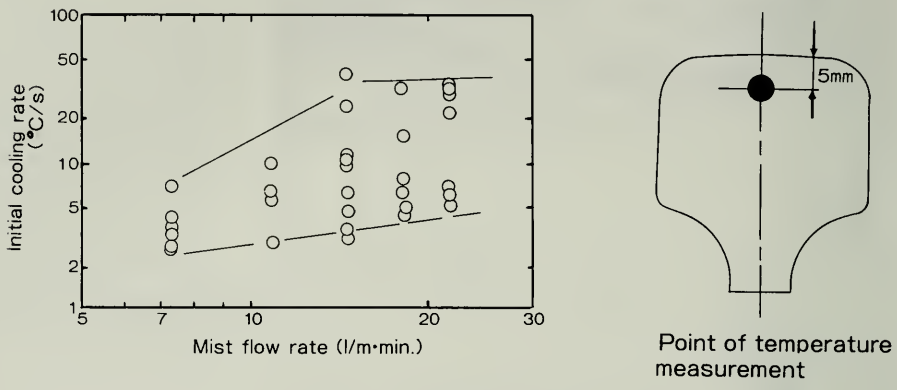


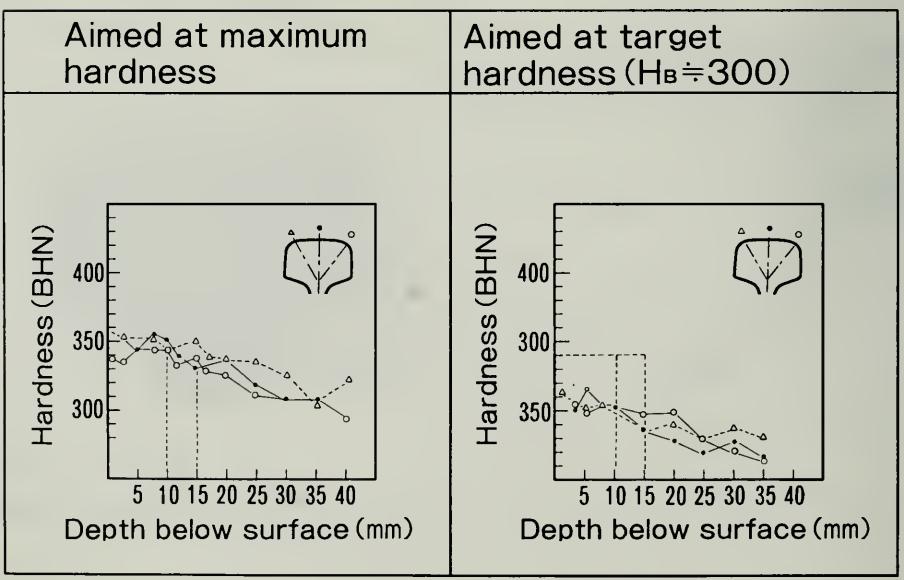
Figure 3. Variation of Cooling Rate in Mist Cooling



c. Results of test by air cooling

The results of an air-cooling test conducted on a standard carbon rail are provided in Figure 4. With air cooling, as is obvious from the test data, it is possible to obtain uniform strength throughout the length but difficult to produce high strength of the NHH class (BHN:350 to 390) in standard carbon

Figure 4. Hardness of Standard Carbon Rail (Test Using Air)



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rails. As illustrated in Figure 5, however, it is possible to insure high strength with air cooling by improving hardenability by adding micro alloy elements.

Figure 5. Hardness of Micro Alloy Steel Rail (Test Using Air)

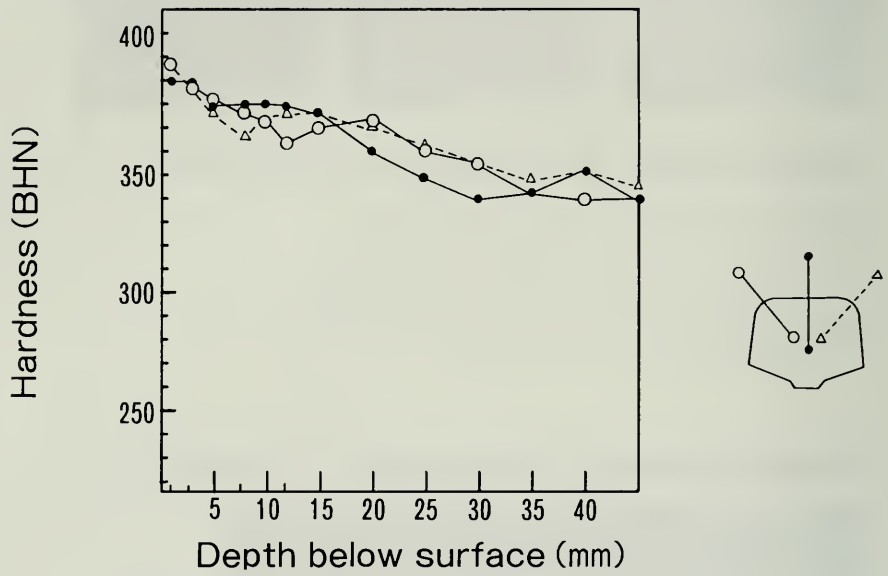
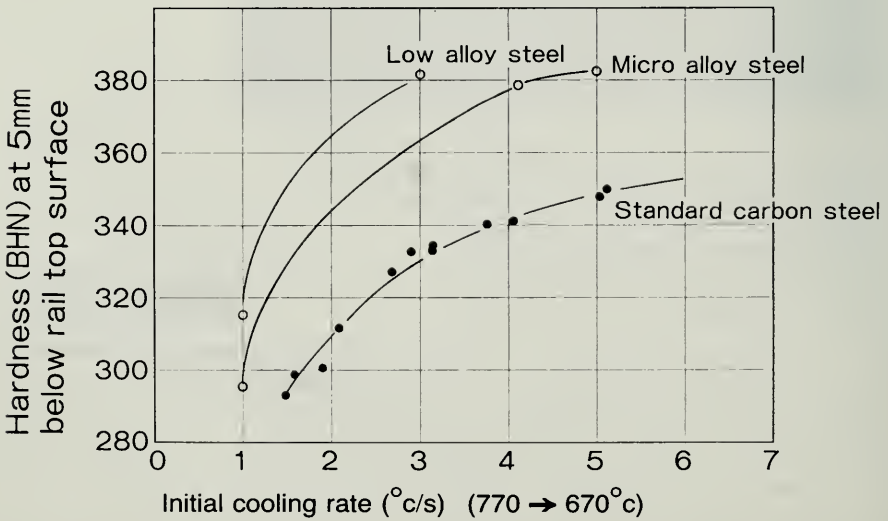


Figure 6 shows a relationship between initial cooling rate and rail head surface hardness.

Based on these test results, NSC has selected air as the coolant for use in the volume production of heat-treated rails of highly stabilized quality.

Figure 6. Relation Between Hardness and Initial Cooling Rate



4. MANUFACTURING OF DHH RAILS

Based on the above-mentioned results, we started construction of Direct Heat Treatment Equipment at the beginning of 1986, and mass production in October 1987.

And we have already shipped 100,000 tons to many countries throughout the world by the end of February 1989.

The features of Direct Heat Treatment Equipment and Products are as follows:

(1) Direct heat treating technology

a. Computer controlled cooling system

The equipment has a computer system which can control automatically the cooling pattern of rails based on the informations of the temperature and chemical composition of the rails.

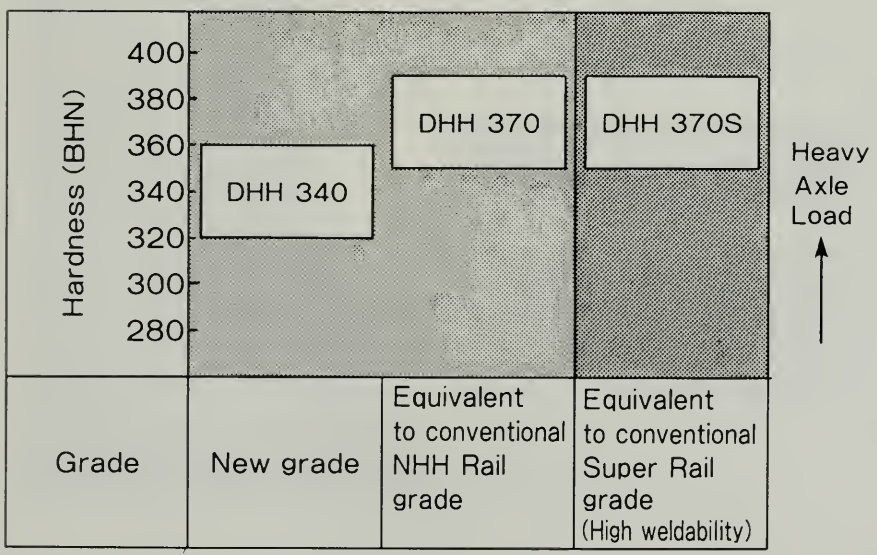
b. Special cooling

The amount of rail bending after cooling is minimized by special cooling.

(2) Product list of DHH rails

NSC named these new heat treated rails "DHH Rails." The product list of DHH rails is shown at Figure 7. The number 340 and 370 indicate the average of brinell hardness number of the rail head surface, and "S" means Super rail with high weldability.

Figure 7. Product List of DHH Rails



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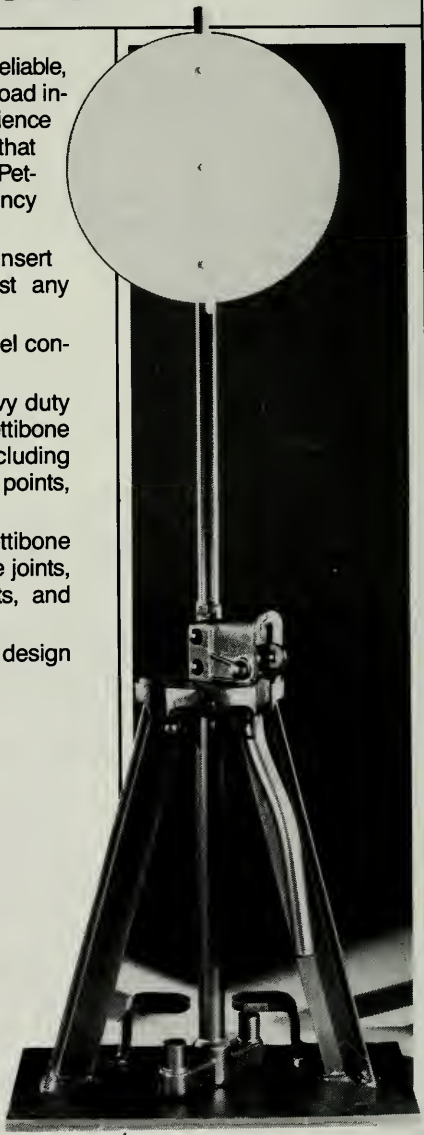

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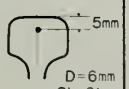
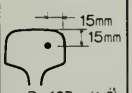
5. PROPERTIES OF DHH RAILS

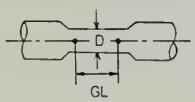
Described below are the principal properties of DHH rails.

(1) Mechanical properties

The mechanical properties of DHH rails are shown in Table 2 in comparison with those of NHH rails. The strength of DHH rails measured at a point near the surface is almost the same as that of conventional NHH rails, but the strength of DHH rails measured at a point further inside is higher than that of NHH rails.

Table 2. Mechanical Properties of DHH370 Rail in Comparison with those of NHH Rail

	DHH 370 rail				NHH rail			
Item / Sampling position	0.2% Proof stress (Mpa)	0.2% Proof stress (Mpa)	Elongation (%)	Reduction of area (%)	0.2% Proof stress (Mpa)	Tensile strength (Mpa)	Elongation (%)	Reduction of area (%)
	863	1,294	15	34	843	1,304	15	35
	834	1,275	12	29	745	1,157	10	25



(2) Hardness in cross section of rail head

Figure 8 shows the cross sections of rail heads of DHH rail and NHH rail. It can be seen that the two types of rails have the same surface hardness but that the DHH rail is hardened deeper than the NHH rail.

(3) Hardness on surface of rail head top

Figure 9 shows the hardness on surface of DHH rail head top. It can be seen that DHH rail has uniform longitudinal hardness distribution of rail head.

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Figure 8. Hardness of DHH370 Rail in Comparison with NHH Rail

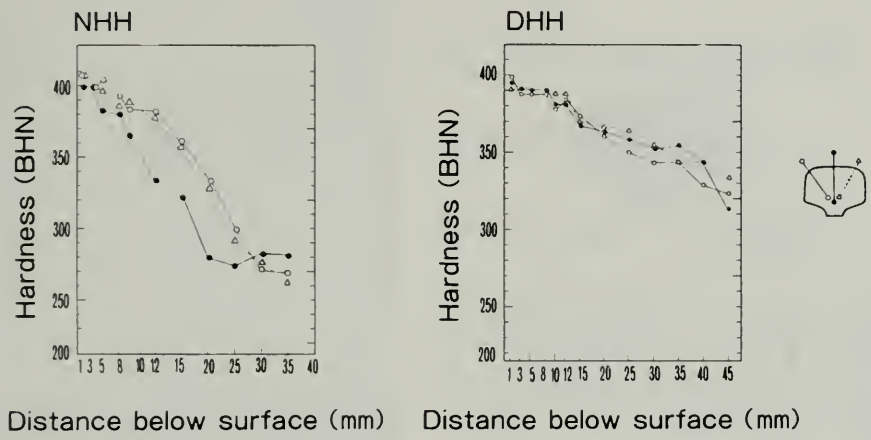
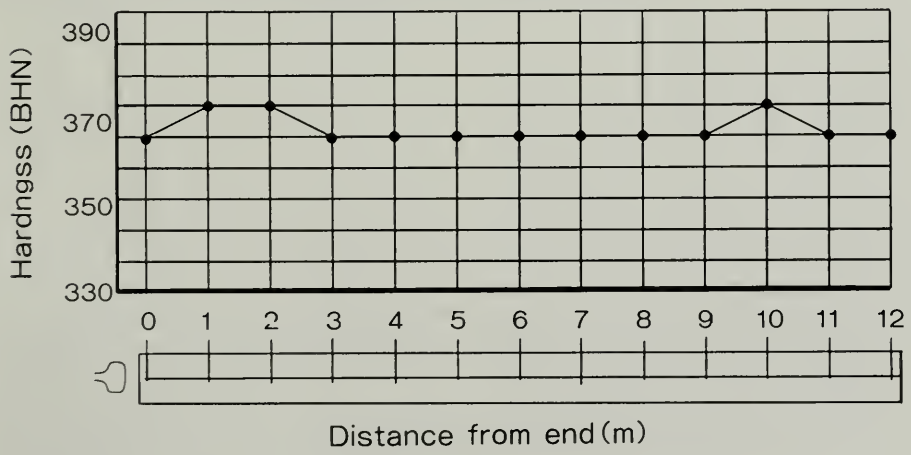


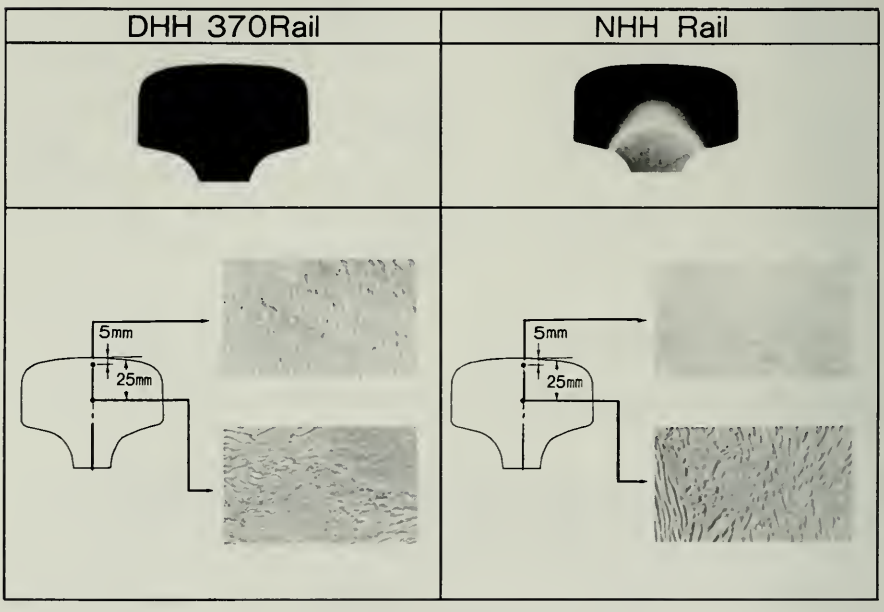
Figure 9. Typical Hardness on Top Surface of Rail Head (DHH370)



(4) Macrostructure and microstructure

Figure 10 shows the macrostructure and microstructure of DHH rail in comparison with NHH rail. The macrostructure of NHH rail shows a softened layer caused by reheating. But DHH rail shows uniform macro-pattern and does not have a softened layer observed. As for microstructure, besides DHH rail has the same fine perlitic structure as NHH rail near the surface at (at 5mm), but DHH rail has fine perlitic structure to a deeper point (at 25mm).

Figure 10. Macrostructure and Microstructure of DHH370 Rail in Comparison with NHH Rail



(5) Wear resistance

Figure 11 shows the results of wear resistance using test pieces taken from the head surface layers of various types of rails. In the case of rails having perlitic structure, harder rails generally show better wear resistance.

This suggests that the DHH rail can retain the wear resistance of its surface layer even when the head surface is ground, while the wear resistance of its surface layer is almost the same as that of the NHH rail as shown in the diagram above.

And Figure 12 shows the results of a wear resistance test using test pieces taken from various depths below the rail head surface. We see that DHH rails have better wear resistance than NHH rails, and that this difference becomes greater the deeper one goes into the rail.

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The Railway Tie Association

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Figure 11. Relation Between Weight Loss by Wear Test and Rail Hardness

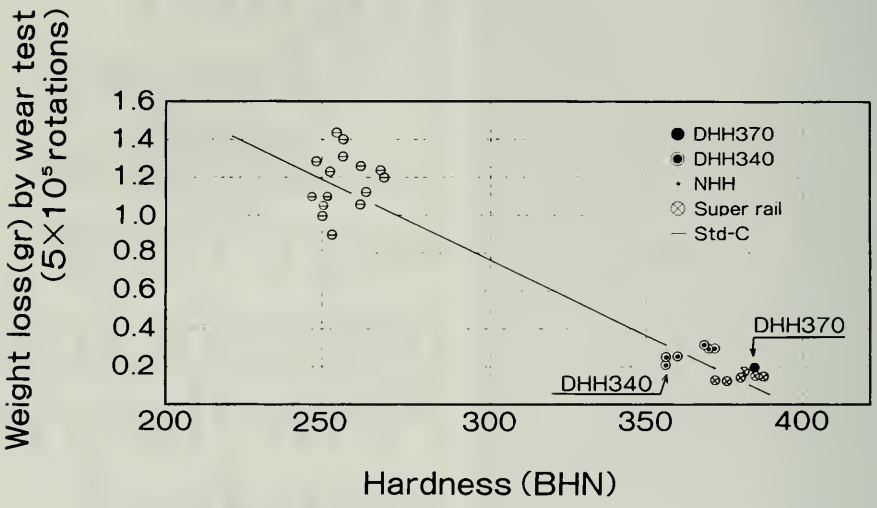
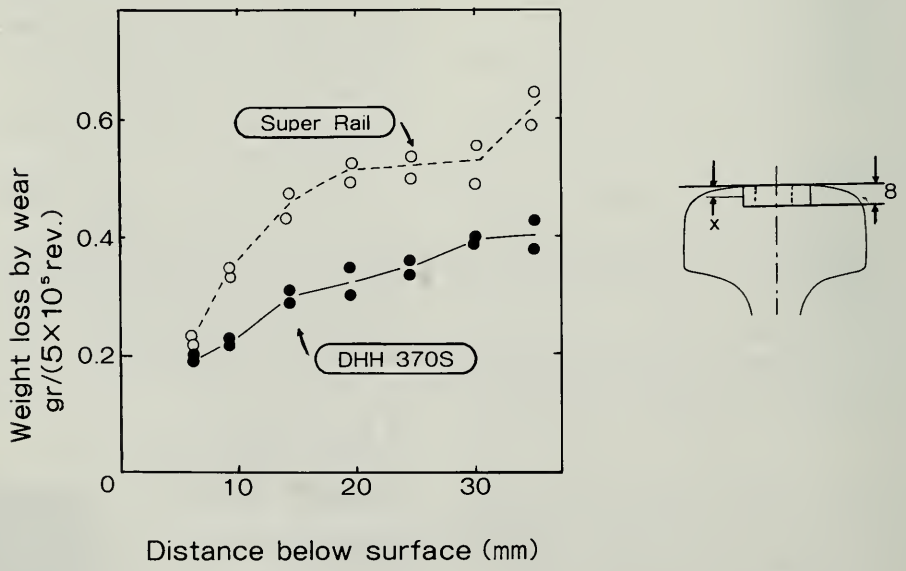


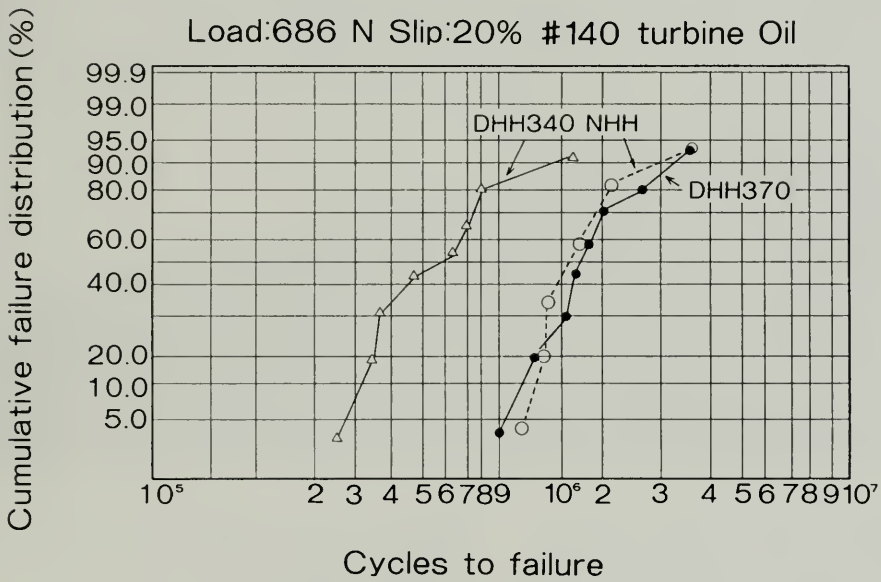
Figure 12. Results of Wear Resistance Test of DHH Rail and NHH Rail



(6) Rolling contact fatigue characteristics

Figure 13 shows the results of a rolling contact fatigue test conducted on DHH and NHH rails. The test was conducted on rails which were oil lubricated to prevent wear. The 370 rail shows an excellent resistance to damage compared to that of the conventional NHH rails. The DHH 340 rails show a good resistance to damage, too.

Figure 13. Rolling Contact Fatigue Test



(7) Residual stress

Figure 14 shows the residual stress in DHH and NHH rails. In the direct heat-treatment process, the amount of rail bending can be minimized by special cooling.

Hence, the reduction through roller straightening can also be minimized. This makes it possible to reduce the surface residual stress in the rail top to near zero.

SHAPING THE FUTURE OF RAIL

The Speno rail maintenance units and the teams of Speno professionals have been a familiar sight on American railroads for decades. The working partnerships that have evolved have led to exceptional progress in the development of more and more precise techniques for the maintenance and the extension of rail life. Most notable, the concept of rail profiling has emerged as the most successful thrust towards solving contemporary rail maintenance problems resulting from ever higher speeds and increasing loads.

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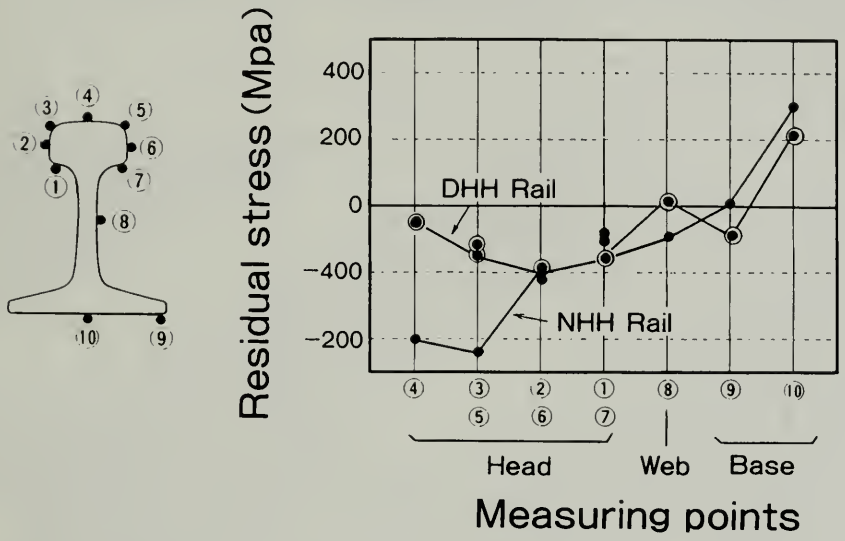
Figure 14. Typical Residual Stress on Rail Surface

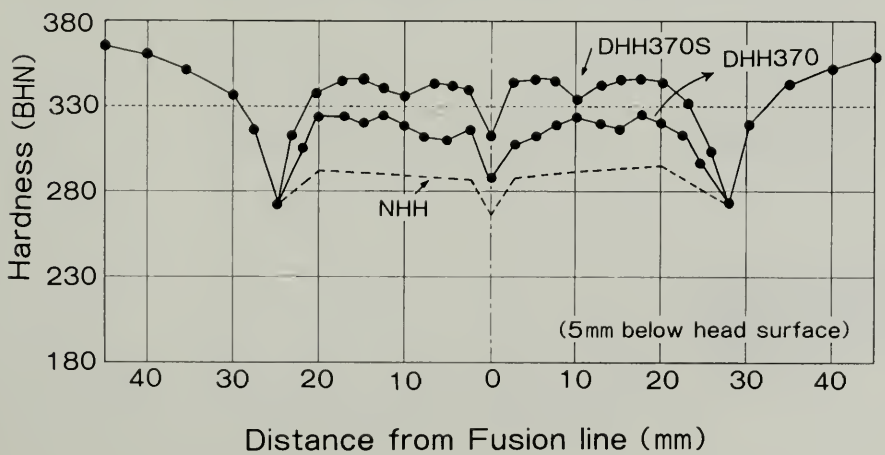


(8) Weldability

Figure 15 shows a hardness distribution of DHH rails subjected to Flash-butt welding. Compared with conventional NHH rails, the DHH rails are subject to less deterioration of hardness of welded section, suggesting that no heat treatment is needed after welding.

The DHH rails which are manufactured by adding small amounts of alloying elements and by controlling the rate of air quenching offer excellent weldability as shown above, thanks to the favorable effect of the alloying elements.

Figure 15. Hardness Distribution on Transverse Section
— Flash Butt Welding (Welder: K355) —



6. CONCLUSION

Nippon Steel Corporation has successfully developed a revolutionary heat-treated rail, DHH rails for short, which are heat-treated directly without reheating.

The features of DHH rails are as follows;

1) Various grade types of strength and hardness suitable for the intended application

2) Deep and uniformly hardened layer in rail head

3) High weldability

4) Uniform quality throughout the entire length

5) Shortening of delivery time

DHH rails are expected to greatly help prolong rail service life and reduce the maintenance costs of railroads.

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9) C. Wesselmann, "—but What's It Doing to The Track?", Modern Railroads, paper 39-40, Mar. 1986.

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ADVANTAGES OF NON-EQUILATERAL AND EQUILATERAL TURNOUTS

By: W. E. Van Trump*

INTRODUCTION

In 1981 the Union Pacific Railroad had 15 #20 equilateral turnouts. Today we have 4 mainline equilateral turnouts remaining.

We feel that the maintenance costs outweigh any operational benefits that we were getting from these turnouts.

The equilateral turnouts we had were used in three basic circumstances:

1. Mainline subdivision junctions.
2. End of double mainline territories.
3. Ends of long sidings (2 to 3 miles long).

The reason the equilaterals were installed in the first place was purely operational — the ability to run trains through both sides of the turnout at high speeds. 60 m.p.h. on original Union Pacific Railroad with 39′ switch points. 50 m.p.h. on Missouri Pacific lines with 30′ switch points.

We looked at the traffic patterns going through the equilaterals a couple of years ago on our Chester Subdivision south of St. Louis where we have a 4-1/2 mile section of single main line CTC territory with double mainline CTC on each end that we call the "Staff." We found a couple of interesting facts. First, we were not running 50% of our trains each way. Because of the priority of trains and dispatcher habits, we were running about 60% of our trains on the east track and 40% on the west. The second interesting thing we discovered was that a significant percentage of the lower priority trains were either slowing down or stopping at one end or the other of the staff in order to allow higher priority trains to go across the staff first. This negated the advantage of having an equilateral in place for these trains, as most of them couldn't make speed through the turnout anyway. We decided to change the two equilaterals out because of the maintenance problems we were having with them, and put in #20 non-equilateral turnouts in their place making the east track the straight side on both switches. We found that there was virtually no negative impact on the train operations as a result of these changes. The additional delay to those trains that could have made 50 m.p.h. through the turnout that were held to 40 m.p.h. because of the change was more than offset by the trains that were allowed to run the subdivision maximum speed through the straight side instead of having to go through the 50 m.p.h. equilateral.

This was not unique to just this location. We found that none of our equilaterals were running an even amount of traffic both directions and that changing the equilaterals to non-equilaterals would not significantly hinder traffic movement.

MAINTENANCE CYCLE

It is our experience that equilaterals take more maintenance time than lateral switches. Every train must run through a turnout when going through an equilateral switch. This constant traffic through a 45 minute curve that we are unable to put elevation in makes it difficult to maintain proper alignment through the switch area and causes gage widening problems due to the lateral forces that are always associated with curves. Photo 1 demonstrates the affects of not being able to elevate curves. Our experience indicates accelerated stock rail and point wear, differential plate cutting, switch plate wear and guard rail wear, as well as gage widening, are all symptomatic of equilateral turnouts. This accelerated wear of all components doesn't allow us to change out the parts of a turnout that historically wear out fastest, like the points and stock rails and utilize the rest of the existing turnout as often as we consider desirable. Rather, we can replace the heavy wear parts only once or twice before it becomes necessary for total turnout rehabilitation. This accelerated turnout maintenance cycle has an obvious negative effect on the bottom line.

*Maintenance Engineer—Track, Union Pacific Railroad

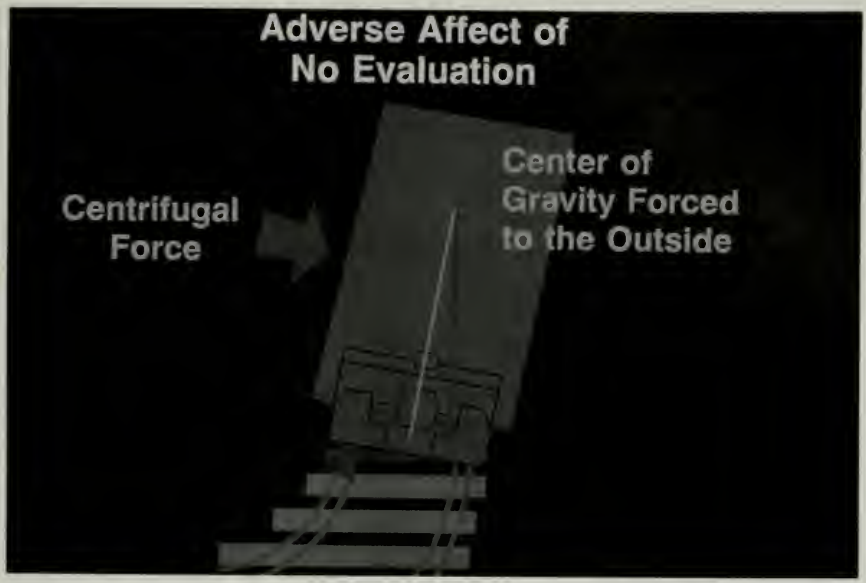

Photo 1

The UPRR has always prided itself on the ride quality of its track. Not just for the aesthetics of nice looking track, but because we believe that it is easier to maintain track that has good ride quality because of the reduction of wear and impact damage that is associated with alignment and profile deviations. We are all aware that minor imperfections in track surface on curved track produces higher dynamic lateral loading than on tangent track. It also produces more pronounced rocking which compounds the dynamic loading problems. Running every train through an equilateral turnout reduces the ability to provide good ride quality, thus creating an unfavorable increase in the maintenance cycle.

The obvious reason given for the installation of equilaterals was the ability to increase the speed through what would be the turnout side of the switch. We fell for this line from our operating brothers so hard that back in the 60's and 70's we installed equilaterals where we had lateral turnouts originally. In order to hit these new turnouts at the proper angle we designed reversing curves (See Photo 2) in advance of the turnout in what had previously been tangent track. So, not only did we put ourselves in a situation where we now ran every train through the turnout side of a switch, we also ran every train through the reversing curves ahead of it; further reducing our ride quality.

CONCLUSION

As a result of our study we reached the following conclusions:

1. Equilateral turnouts require increased maintenance.
2. Equilateral turnouts require additional inventory.
3. Equilateral turnouts reduce ride quality.
4. Equilateral turnouts cause us to reduce our 60 and 70 m.p.h. trains to 50 m.p.h., while not appreciably affecting our slower, lower priority trains.

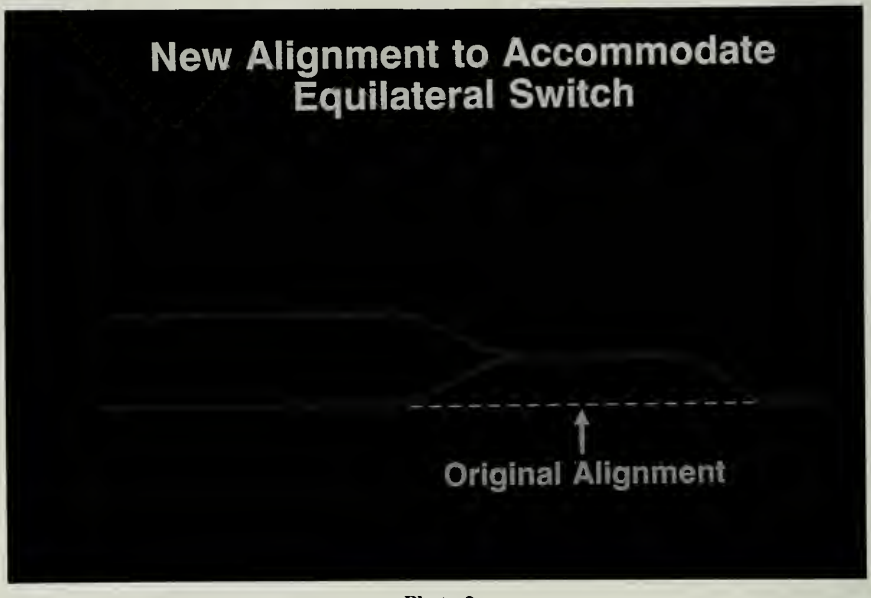

Photo 2

As a result of these conclusions we have decided that equilateral turnouts are not generally desirable for our Railroad, which is why we are changing them out.

Equilateral turnouts seem to be a case where good theory doesn't always pan out in practical application. Careful study of traffic patterns and needs should be made when considering the feasibility of equilateral turnouts.

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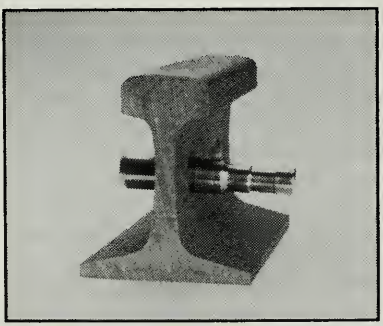

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USE OF TWO "HIGH SPEED" SCALES IN TANDEM

By: K. E. Bruestle*

INTRODUCTION

Over the years Burlington Northern, like many other railroads, has wrestled with the inefficiencies of weighing railroad cars on our conventional track scales in the conventional manner. Weighing on static scales where extensive switching is involved is very costly and inefficient. But, a certain amount of weighing just has to be done somewhere by somebody in the railroad industry. The law requires it where there are no working weight agreements with the customer. Failure of the originating road to weigh cars so designated, "To be Weighed," will result in interchange back charges by the receiving carrier under the terms of Car Service Rule 11.

Detection of overloads, has been, and is increasingly so, a much-needed capability. Some roads refuse to take an overloaded car in interchange traffic as a matter of policy. Overloads have caught the attention of railroad Mechanical Departments, to the extent of checking for some correlation between overloaded roller bearings and derailments. If there is a correlation, and a suitable means of detecting overloads, the potential for savings and safety improvements is sizable. To pursue the overload issue further, railroad Bridge Engineers have reason for concern, also. Unit trains of coal or grain are delivering cumulative fatigue damage to our bridges, especially the shorter spans, and the older structures of lighter design. Better knowledge of the actual axle loads is becoming imperative. The advent of the articulated container car has produced some headaches for Bridge Engineers and Track Engineers, too. The probability of getting very large overloads on the interior trucks of these units is quite high. These heavy loads must be detected to avoid damage to our track and structures.

American railroads are losing revenue by hauling more lading than they are being paid for. The capability to weigh accurately with a minimum of interference with the normal flow of traffic, presents a potential for enormous savings to the railroad industry and to the consumer. Thus, we at Burlington Northern perceived the need for a high speed, coupled-in-motion, certifiable, main line track scale. This need to have a better knowledge of what we are hauling, and to minimize the cost of gaining that knowledge became an obsession at BN in 1985. That obsession started a study which led to the topic on which I am talking today.

DESIGN

Having made the decision to install a new scale was the easy part of this project. For Burlington Northern, defining the design criteria and choosing a location for such a scale turned out to be a very large undertaking. From our experience, we were breaking new ground. Our goal was to get our proposed scale certified for mixed traffic at the highest speed attainable by present technology. A study was started to determine limiting grades and track curvature, and the effects on weighing accuracies, of switches in close proximity. It was soon determined that there was not a lot of literature available on this subject, even though several other railroads either were considering coupled-in-motion scales, or had installed one or more already. An integral part of the preliminary study must, of course, include a traffic density review for determination of optimum locations for scale installations. The scale must be placed where it will do the most good in your overall operations.

The two significant sources of error when weighing coupled-in-motion railroad cars are conversion of coupler forces into vertical components, and dynamic components superimposed on the static weight. To minimize these sources of error, very serious consideration would have to be given to the approach slabs, rail fastenings and track quality and geometry adjacent to the scale. It was felt that the better the track quality and approach stiffness, the better would be our chances of attaining accurate weights. The challenge in designing this scale installation, then would be to pay enough attention to the approach conditions to be able to obtain certification, but without spending money needlessly on overkill. The importance of accurate weights cannot be over emphasized. To get our scale certified, we would be required to comply with the provisions of Handbook 44 of the National Bureau of Standards,

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and be able to weigh within 0.1% accuracy. For bi-directional weighing, we determined that a level grade with tangent alignment for one train length on either side of the scale location should be the ideal situation. A study of our track charts revealed that we did, in fact, have several such ideal locations on our railroad, but almost invariably, there was some detraction from the ideal situation which did not show up on the track charts. Additionally, most Operating Department people did not want to be the guinea pigs, and go through the disruptions of allowing such an installation on their territory and then not be able to get the scale certified. For various of the reasons cited, we decided that for this experiment, we would go with weighing in one direction, only. A tentative sight was located in Kansas City and design for that location was started.

At about this time, it was suggested that since this project was considered somewhat experimental, why not install and test more than one scale. Certainly, the cost of constructing and testing two scales at one location would be a lot less than the cost of two separate facilities, and with the two scales installed in tandem, we would learn a great deal about scale capabilities in half the time. The decision was made

Photo 1

to use a deep pit, weigh bridge, load cell, truck-weighing type scale in tandem with a track level, axle weighing unit (Photo 1) placed in advance of it. Plans were developed to place the pit scale in a relocated track, with 300′ of reinforced concrete approach slab and 75′ of departure slab. The track level scale would be located in the approach slab, near its center. These approach and departure slab lengths were felt to be adequate to stabilize the cars sufficiently to reduce car dynamics to a minimum. The approach to the first scale would include about 3200′ of tangent track, slightly less than ideal, but nonetheless a good stretch of tangent. The grade across the scales, and for about 2000′ in advance, was set at 0.17% descending. A 0.17% grade was chosen because it was determined that this slope would reduce the effects of rolling friction, but would not cause free roll, thereby minimizing coupler forces.

Considerable grading was required in the approach track to attain the specified grade as seen in Photo 2. To conform to this geometric configuration would require constructing 900′ of new track with 115# rail, and lining over and raising about a half mile of existing track. Since all of these design criteria could be met without incurring unreasonable costs, the decision was made to go ahead with the project.

Photo 2

CONSTRUCTION

The project was bid in late 1987, with the pit scale supplier acting as prime contractor to construct the pit in which his scale would be installed, and the approach slab and exit slab as well. Each scale manufacturer was then responsible for installation of his own scale. BN would handle the approach grading and related track work. Construction began, then in the Spring of 1988.

Soil borings taken subsequent to the letting of the contract indicated that piling would be required to properly support the reinforced concrete pit and the approach and exit slabs. Augercast piling were chosen for this location. The piling installation was added to the contract and was phased in with the rest of the foundation work.

As in any old rail yard, you never know what you may encounter when you open up the ground, and this was no exception. We discovered a 15″ water line crossing our approach slab at a flat skew, necessitating a redesign of the pile layout and causing some anxieties in the meantime. The NCAA basketball championship playoffs were underway in nearby Kemper Auditorium, and it was thought that this water line was the main service to that facility. Happily, we were able to complete this phase of the foundation work without disrupting the Final Four at Kemper. Had we chosen driven piling, I might be telling you a different story today.

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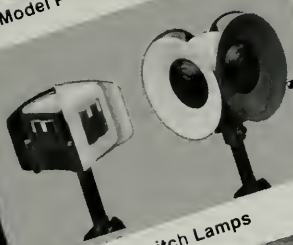

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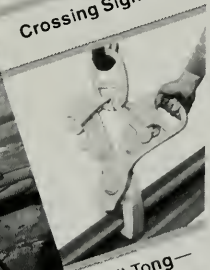

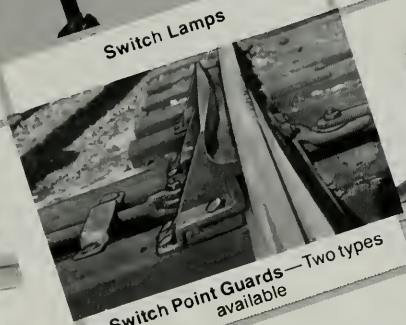

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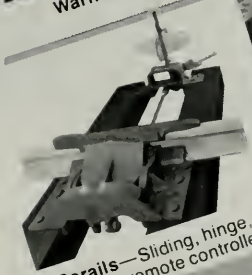

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The pit depth (Photo 3) required that the excavation be extensively shored to protect adjacent live tracks. In future installations we may look at ways to minimize the pit depth, and thus reduce shoring needs. Foundation work continued throughout the summer, with very little disruption due to weather, because of the severe drought we experienced in the Kansas City area last year. By fall, the scales were completely installed and ready for testing. The installation included both scales, the approach and exit slabs, a personal computer, with a printer and other assorted hardware. The computer and support equipment were housed in a vandal resistant scale house. In discussing the aspect of security for our weighing systems with other railroads, it is certainly agreed that this part of the system does justify considerable attention. These facilities are nearly always in remote, poorly lighted locations in which theft and vandalism are more apt to occur. The pit scale system consists of a 20′ long steel weighbridge supported by four 200,000# load cells which feed a 9200 in-motion weighing instrument. This instrument feeds the PC. This PC is then further linked to a PC at the Yard Office.

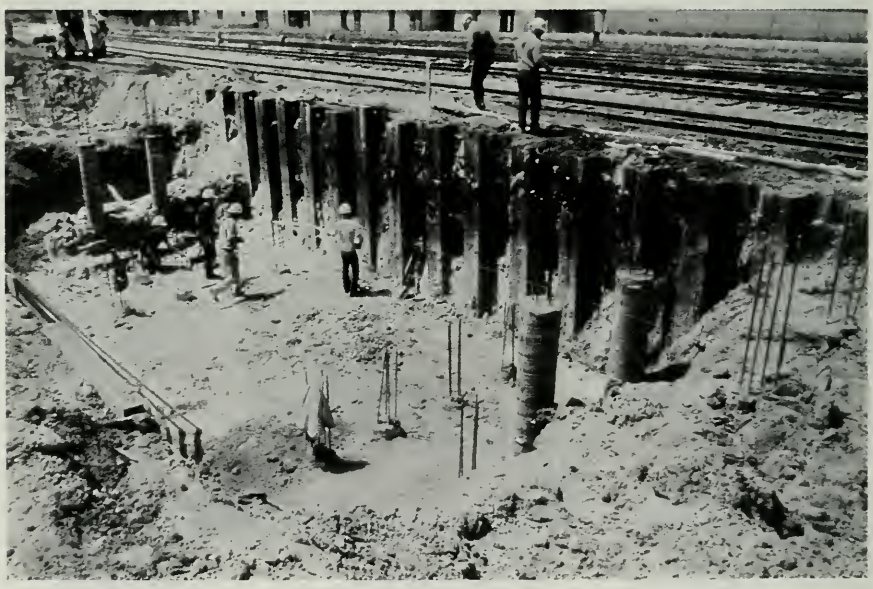

Photo 3

Some of the other features of our tandem scale installation are:

Mitered rail ends and wheel detectors for the pit scale.
Rail creep restrainer on the pit scale.
Wheel detector for the axle weigher.
Grout pad detail used to achieve proper rail elevations on the approaches.

TESTING

Testing of the completed facility began by checking weighing accuracies against a certified 100,000# BN test car, positioned statically. When the necessary adjustments and calibrations on the two scales had been made to assure accurate static weighing, we were ready to begin dynamic testing.

For testing purposes, we made up a set of 10 monitor cars at our Havelock, Nebraska shops. These monitor cars were made up in groups of two each, with two empties, two at 100,000#, two at

150,000#, and two at 250,000#. When work on the monitor cars was completed, they were weighed on the Master Scale in Havelock and shipped to Kansas City to begin dynamic testing.

The testing started with a series of runs across the scales at very low speeds with a captive switch engine and the monitors placed in various sequences. After making any initial adjustments to the scales, if required, the test cars were inserted in actual revenue trains as they passed through the yard. With successive train movements, the block of cars was placed in different positions in the trains to obtain data under the varying conditions. In addition, as the load configurations were varied, so were the operating speeds, up to 15 miles per hour.

RESULTS

After thoroughly subjecting our new facility to this strenuous testing, the State of Missouri certified one of these scales, the load cell type, for coupled-in-motion, commercial weighing at speeds up to 12 miles per hour. The axle weighing scale is not yet certified by the State, but the manufacturer replaced several components in the scale in late February, and we are presently testing again. We are guardedly optimistic that we will soon receive State certification on this unit, also.

In normal operation, 450 to 475 cars will be weighed on a daily basis, including both loads and empties. Every car type is being weighed at this location.

FUTURE PLANS

We have budgeted for another high speed coupled-in-motion track scale for 1989 at a site yet to be chosen near Denver, Colorado. This will not be a tandem installation. This is part of an overall plan to one day have as many as eight scale locations. With this array of scales, we feel that we will be able to weigh the vast majority of those shipments that require weighing on Burlington Northern.

Some changes in the design which may be considered in future installations include the following:

- — Addition of Dead Rails. Use of dead rails will be considered where normal track speed at a particular location is 40 to 60 mph. This will give us the choice, then, of running some of our trains over the dead rails, so as not to subject the scale itself to unnecessary abuse. Also, repairs and maintenance could be made under traffic without upsetting train operations.
- — Heavier Design. A heavier design may insure greater durability when running trains across the scale at normal track speed, although, at this point we have seen no signs of abuse to our Kansas City facility.
- — Shallower Pit, where a pit scale is our choice. There are some tradeoffs to be considered here, especially where shoring requirements are severe.
- — Rail Restraint. We may consider use of rail anchors on the approach rails. Adequate rail restraint is vitally important to proper functioning of the mitered rails, and in turn, accurate weighing.
- — Dual Circuitry. This feature gives one a check on how the primary circuit is functioning, and provides a backup circuit on those occasions when the primary one may be malfunctioning.

In conclusion, then, from our Kansas City experiment, we at Burlington Northern hope to have learned enough to intelligently plan and implement a weighing program that will address those issues herein mentioned, at least overall cost, and within legal requirements. We intend to accomplish one or two installations a year, as funds permit. This schedule should bring our program to completion, as presently planned, by 1992 or 1993.

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COMMITTEE 4—RAIL

Chairman: A. W. Worth

Report of Subcommittee 3
Rail Statistics

Subcommittee Chairman: A. E. Shaw, Jr.

Consolidated Report of Rail Shipped To North American Railroads from North American and Non-North American Producing Mills in 1987
By Weight and Section

Weight	Section	N. American Tons Shipped	Non-N. American Tons Shipped	Total	% Total
100	RA	0	0	0	0
100	RE	4,869	0	4,869	0.7
115	RE	156,694	26,555	183,249	26.1
119	RE	5,066	0	5,066	0.7
122	CB	2,225	0	2,225	0.3
132	RE	109,515	59,634	169,149	24.1
133	RE	61,656	3,420	65,076	9.3
136*	RE	183,125	83,300	266,425	37.9
140	RE	5,958	0	5,958	0.8
OTHER		0	429	429	0.1
TOTAL		529,108	173,338	702,446	100

*Includes 136# with modified head profile

COMMITTEE 22—ECONOMICS OF RAILWAY CONSTRUCTION AND MAINTENANCE

Chairman: W. C. Thompson

ECONOMICS OF METHODS TO DISPOSE OF SCRAP AND OBSOLETE MATERIAL

Subcommittee Chairman: R. W. Simmons

The numerous and various economics of methods to gather, sort, reuse, and/or dispose of other Track Material, (O.T.M.), are unique to the railroad maintenance-of-way industry. No other industry has a similar material handling, classification, reuse, or disposal problem of a comparable magnitude.

These maintenance-of-way materials, for this report, are included in the I.C.C. Account—Other Track Material (O.T.M.). O.T.M. includes track fastenings—Tie Plates, Spikes, Rail, Anchors, Joint Bars, Track Bolts, Nut Locks, Gage Rods, Derails, Bumper Blocks, Expansion Joints, Compromise Joints, and so on.

For a track maintenance, tie renewal or rail renewal project, the other track material must be picked up and the roadway left clean and free of obstructions. Some railroads have a clean-up crew and work train follow immediately behind the maintenance, tie renewal, or relay project so that the clean-up is entirely completed by the end of the day. This is no doubt the best policy when sufficient work trains, equipment, and people are available and clean-up can be accomplished without excessive overtime.

The problems of keeping other track material separate or the mixing of various sizes or types of material in the field and loading them for shipment are difficult and require good management and planning to be efficient.

The opportunities for savings to the railroad maintenance-of-way management, by the efficient handling and planning methods and making the material available for reuse are many. The cost of inefficient planning and handling and the loss of reusable material along the right-of-way or to the scrap pile can be very great. The loss or savings is magnified in view of today's ever increasing material prices.

The O.T.M. to which this report is directed, is that which is generated in a track maintenance, tie renewal or rail renewal project, or other maintenance programs. Either program or project can generate large quantities of other track material, such as spikes, rail anchors, and ties plates, for example in a rail renewal project.

Currently, the handling of O.T.M. can be separated into four phases;

1. Piling the material along the track and right-of-way for loading into rail cars or trucks by magnet or loader.
2. Picking up and loading the material into rail cars or trucks.
3. Sorting and classification of the material.
4. Disposing of scrap and/or obsolete materials.

These four phases can be reviewed further as follows;

1. **Piling the material along the track and right-of-way for loading into rail cars or trucks by magnet or loader.**

Most railroads use either push cars, onto which they hand-load the material and dump it off at intervals, or a conveyor equipped scrap loader with the associated dump containers. This process would enable them to keep some kinds of materials separate from other kinds, as required. In the case of tie plate pickup, manual piling of the plates, as they are removed, is one approach. Some railroads use a crane with a magnet to set the rail out and then pick up the plates and pile them on the shoulder to eliminate the manual handling of plates.

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EFFICIENCY

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ECONOMY OF LABOR

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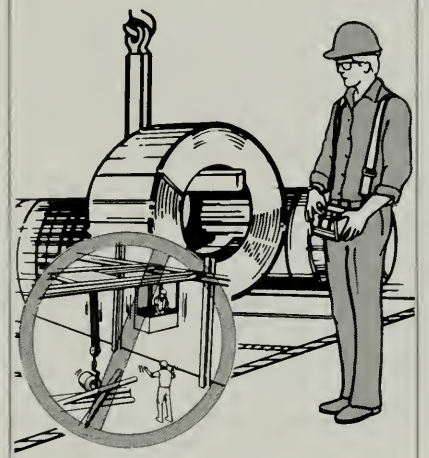

SAFETY

When using radio remote control (R/C), the R/C operator is in complete control. There are no misinterpreted hand signals, (a major cause of accidents with cab operated cranes) or height and clearance questions concerning the load.

EFFICIENCY

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ECONOMY OF LABOR

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2. **Picking up and loading the material into rail cars or trucks.**

The common practice is to load the piles of O.T.M. material with a crane equipped with a magnet. The on-track crane and gondola cars is the most often combination.

For major rail projects, the pickup work is done immediately following the rail change to get the benefit of track time afforded by the rail change. An off track crane and trucks, is used where truck access is possible. The trucks haul the material to the nearest siding, where it is transferred into rail cars. The geography of the separate locations would determine where this alternate method would be most practical. In most cases, a work train with one or more cranes is used. It can easily be seen that the decision of how many kinds of material will be kept separate, and how many will be mixed together, depends on the following;

a. The resources available for sorting the material at some other location.
b. The track time available for the pickup operation.
c. The quality of the O.T.M. material and the need for keeping it separate. If all the O.T.M. material is scrap value only, it does not need to be kept separate.

3. **Sorting of the track material.**

Separating the reusable O.T.M. material from the scrap or from the material that is to be rehabilitated, is handled quite differently on some railroads. Two schools of thought become evident. The first school claims that sorting should be done in the field to simplify the handling problems downstream. The other school claims that all the material should be loaded together and sent to a central depot for processing where conveyors, trained personnel, automated packaging, and material handling can make the operation more efficient. For the railroads that use the central depot approach, we find that some have their own facilities on the property, while others use an outside contractor to perform the service for a fee.

Some of the advantages of a centralized facility include the following:

a. It provides for better working conditions and thus increases worker productivity.
b. Better quality control is achieved because it is easier to train and supervise material classifiers. The quality of recoverable materials will be more uniform, with better quality control of the operation.
c. A sheltered area with good lighting enables workers to better gauge and identify material.
d. Mass-production operation enables a more efficient approach to containerizing, bagging, or packing the materials with proper labeling.
e. The accuracy of classification in a properly managed operation leads to confidence by the user of the material and, thus, increases the probability of actual savings by reuse, rather than purchase of new material. If carelessly classified, it may end up as scrap by the final user even after it was processed.

One solution does not fit all conditions. Depending upon the quality of the material being generated, it may be better to pre-sort some of the material as it is being removed to keep it separate from other small material that may have to be classified for disposal or reuse. Because of this variation of quality, good judgment and sound management on the part of the field supervision are essential. Only through good judgment can the proper decisions be made to minimize waste and make the entire operation economically efficient.

In summary, the following conditions need to be analyzed in an engineering and economic analysis of the pickup of O.T.M. from a track maintenance, tie renewal, or rail renewal program. This analysis would include all labor, equipment, fixed facility and overhead costing, to arrive at the most economical and feasible method of gathering all excess material from one of the projects.

1. All O.T.M. can be loaded together and shipped from a project to a central location.

2. All O.T.M. can be semi-sorted on the project into different types of material, and then shipped.

3. All O.T.M. can be completely sorted as to type and then shipped.

4. All O.T.M. can be completely sorted and classified and then shipped.

As mentioned previously, a complete engineering, economic and cost analysis must be performed to determine which method is most cost beneficial to each individual railroad.

For a centralized material handling depot, many options are available to the railroad maintenance-of-way management. A railroad may have their own centralized depot, or more than one depending upon their individual operations and locations, staffed by their own personnel.

Another option, would be to have their own depot, staffed by a contractor for the service. In addition, a total outside company could be contracted with, in many different ways, a fee paid for sorting and classifying the material, or all material could be sold to the company and then the reusable material repurchased by the railroad.

4. **Disposing of scrap and/or obsolete material.**

Disposing of scrap or obsolete materials, i.e. Other Track Materials may be done in different ways. Obsolete materials would first be offered to the numerous track material supply companies, either by closed bid or through negotiations, assuring that all prices obtained were more than current scrap value.

All scrap track material is normally handled by the Stores Department on most railroads. However, the selling of "Scrap" is a science unto itself. A complete knowledge of the scrap metal industry is a *must*. Scrap metal prices vary from day to day and month to month, as well as to location. Railroad personnel with thorough knowledge of the scrap metal industry can be of considerable economical benefit to the individual railroads, and is an area which can be capitalized on for the financial benefit of each railroad. At times it may be beneficial to hold a large stock of scrap material, rather than to sell at the current scrap prices.

The sortation, classification and reuse of other track material, also requires consideration. An estimate of the current pricing of new track material can be found in the monthly market quotations by ENR field reporters, "Engineering News Record" magazine. Additionally, all sizes of new track material are not currently available, with many railroads, each having their own designs and specifications. The value of salvageable and reusable material may be even higher than new material, where the material is obsolete and is not available through normal suppliers.

In general the following observations can be made relative to reusable O.T.M. from a track maintenance project;

1. Track Spikes.

   Track spikes can be reused as-is, if unworn, straight and clean. Some railroads will clean the track spikes in a tumbling operation, while others will straighten them for reuse.

2. Tie Plates.

   Tie plates can be reused if in good condition and straight. In some cases the tie plates can be repunched, if single shoulder, for different spiking patterns and rail base sizes.

3. Track Bolts.

   Track bolts can be reused if straight, have a full section, and will allow the nut to be hand-threaded two full turns. Soaking in oil is common, but rethreading is not a general practice because they may not stay tight after rethreading.

4. Rail Anchors.

   Rail anchors can be reused in some cases. Some railroads will reform the anchors. This of course, requires proper quality control measures.

5. Joint Bars.

   Joint bars are generally reused or reformed after careful inspection to insure that cracked bars are not reused. A magnetic particle inspection procedure may be used to make certain that no cracked bars are allowed to slip through inspection.

The procedures for making economic comparisons between O.T.M., either new, relay or the various classifications of rehabilitated material may be made by the following procedures;

Capital Cost Method[1]

To compute comparative capital costs, use is made of the compound-interest formula $c = c'(1 + r)^n$, where c is the first cost of the individual track material item, or, in other words, the total amount of principal plus the accrued interest at the end of n years, c' is the principal or original dollar investment per individual track material item, and r is the interest rate. The yield c must be large enough to cover the cost of the first track material item and still leave a sum that would again reproduce enough to buy another track material item at the end of n years at a rate of interest r. In other words,

$$c + c' = c'(1 + r)^n$$

$$c = c'(1 + r)^n - c' \text{ or } c = \frac{c}{(1 + r)^n - 1}$$

and c' is the amount that must be set aside to make the individual track material item life perpetual. Since, however, the initial track material item purchase must be made, the total capitalization is

$$c + c' = c \frac{(1 + r)^n}{(1 + r)^n - 1}$$

Annual Cost Method[1]

As opposed to the capital investment procedure, an individual track material item cost may be compared on an annual basis whereby provision for replacement is made by allocating a certain sum each year to a sinking fund, which after n years at compound interest will provide the amount, interest and principal, that will cover the cost of a new or replacement track material item. The lowest cost or cheapest individual track material item is one for which interest on the first cost plus the annual sinking fund charge is a minimum.

Again c can be the first cost of the individual track material item, r the interest rate, I the interest rate on the first cost, and A the annual charge at compound interest that at the end of n years will provide for a replacement track material item. Annual interest on first cost will be $I = cr$. The amount of the sinking fund must equal the first cost of the track material item after n years or, in terms of the preceding values,

$$c = c'[(1 + r)^n - 1]$$

But $c' = A/r$. Then

$$c = (A/r)[(1 + r)^n - 1] \text{ and } A = \frac{cr}{(1 + r)^n - 1}$$

The total annual cost will be

$$I + A = cr + \frac{cr}{(1 + r)^n - 1} = \frac{cr(1 + r)^n}{(1 + r)^n - 1}$$

[1]American Railway Engineering Association, "AREA Manual for Railway Engineering," Chapter 22, Section 2.2, pages 22-2-5 to 22-2-7, 1987.

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The individual track material item costs may be considered equivalent when the annual costs or the capitalization are equal or when

$$c' = \frac{c\,(1 + r)}{(1 + r)^n - 1} \times \frac{(1 + r)^{n'} - 1}{(1 + r)^{n'}}$$

where r is still the interest rate, c is the cost of the individual track material item with n years of life, and c' is the cost of the individual track material item of n' years of life. The foregoing analyses are based on those recommended by the A.R.E.A.

EXAMPLES IN USE OF METHODS OF ANALYSIS

EXAMPLE 1

Capital Cost Method of comparison for a new and relay tie plate. For this comparison an A.R.E.A., Plan No. 12, eight hole punch plate is used. All freight charges have been excluded.

1. New Tie Plate
   a. Weight is 21.47 lbs.
   b. Cost of new tie plate, is taken from "Engineering News Record" magazine, ENR Material Price, June 16, 1988, page 62.

      Price \$28.10/C.W.T. x 0.2147 = \$6.03 Each

   c. c' = Original dollar investment per plate = \$6.03 Each

      r = Interest rate—for example purpose = 0.10

      n = Useful life—for example purpose = 30 years.

      $c = c'(1 + r)^n$

      $c = \$6.03\ (1 + .10)^{30}$

      c = **\$105.22** Total amount or principal plus the accrued interest at the end of 30 years.

2. Relay Tie Plate
   a. For example purposes, the value of a relay tie plate is taken at 50% of the cost of a new tie plate or \$3.02 each.

   a. c' = \$3.02
      r = 0.10
      n = For example purposes = 20 years
      c = **\$20.32** Total amount or principal plus the accrued interest at the end of 20 years.

EXAMPLE 2

Annual Cost Method of comparison for a new and relay tie plate. Same tie plates are used for comparison as in Example 1.

1. New Tie Plate

a. c = First cost of tie plate = \$6.03

r = Interest rate—for example purpose = 0.10

I = Interest on first cost—for example purpose = 0.10

A = The annual charge at compound interest that at the end of n year will provide for a new tie plate

n = Useful life—for example purposes = 30 years

$I = cr$

I = (\$6.03) (0.10)

I = \$0.603

$$A = \frac{cr}{(1 + r)^r - 1}$$

$$A = \frac{\$0.603}{(1 + .10)^{30} - 1}$$

A = \$0.037

$I + A$ = **\$0.64** Total annual cost for a 30 year life.

2. Relay Tie Plate

a. Same tie plate as used for comparison in Example 1.

b. c = \$3.02

r = 0.10

I = 0.10

n = 20 years

A

$I = cr$

I = (\$3.02) (0.10)

I = \$0.302

$$A = \frac{cr}{(1 + .10)^{20} - 1}$$

A = \$0.053

$I + A$ = **\$0.355** Total annual cost for a 20 year life.

Respectfully Submitted,

AREA Committee 22
Subcommittee D-1-88

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COMMITTEE 24—ENGINEERING EDUCATION

Chairman: J. W. Orrison

Report of Subcommittee C1-1-82: Recruiting

Subcommittee Chairman: L. J. Hoffman

A survey of MW&S Chief Engineers concerning college graduates hired in 1988 has been completed. Replies were received from 18 of the 20 railroads requested to furnish information.

During 1988 there were 18 graduates employed compared to eight during 1987.

Table 1 summarizes the type of degree and major course of study for 18 newly employed graduates.

Table 2 is a summary of schools represented by the graduates employed.

During 1988, six of 18 responding railroads employed at least one graduate. One graduate was employed by one railroad, two graduates were employed by one railroad, three graduates by one railroad and four graduates by three railroads. The average number employed by hiring railroads was 3.0. Five of the graduates hired had prior experience.

Employment of Electrical Engineers decreased from six in 1987, to three in 1988. Civil Engineers increased from one in 1987, to 13 in 1988.

The average monthly salary of the 18 graduates employed is provided in Table 3. Salaries reported by U.S. railroads included a high of $3,465 per month and a low of $2,250 per month. Of the railroads hiring graduates, one paid all graduates the same salary regardless of experience.

Co-Op student programs were provided by five railroads with the companies sponsoring 26 students in 1988. The sponsoring railroads paid salaries ranging from $1,375 per month to $2,000 per month. All railroads sponsoring more than one Co-Op student selected from two or more universities. Salary information is shown in Table 3.

Summer employment was provided for three students by one railroad. Salary information is shown in Table 3.

Table 4 is a summary of schools represented by Co-Op or summer students employed.

Table 1

Degrees and Major Courses of College Graduates Employed by Railroads

	Number of Graduates						1988 Distribution	
Degree	**1983**	**1984**	**1985**	**1986**	**1987**	**1988**	**U.S.**	**CA.**
B.S.	30	84	45	43	7	17	17	0
M.S.	1	5	4	0	0	0	0	0
B.A.	0	1	1	1	1	0	0	0
A.S.	0	0	0	0	0	1	1	0
Total	31	90	50	44	8	18	18	0

Major Courses of Study

	Number of Graduates						1988 Distribution	
Degree	**1983**	**1984**	**1985**	**1986**	**1987**	**1988**	**U.S.**	**CA.**
Civil Engr.	22	57	45	23	1	13	13	0
Elec. Engr.	3	15	1	12	6	3	3	0
Business	0	4	1	1	0	0	0	0
Engr. Tech.	1	3	1	1	1	0	0	0
Const. Engr.	2	3	1	3	0	0	0	0
Transportation	0	1	1	1	0	0	0	0
Engr. Physics	0	0	0	0	0	1	1	0
Comp. Science	0	0	0	0	0	1	1	0
Other	3	7	0	3	0	0	0	0
Total	31	90	50	44	8	18	18	0

Table 2

Schools of College Graduates Employed by Railroads in 1988

Youngstown State University	1
Ohio University - Athens	1
Northeastern University	1
Lehigh University	1
Iowa State University of Science & Technology	1
University of Tennessee	1
Purdue University	1
Polytechnic University	1
Rutgers University	1
Queens College	1
Virginia Polytechnic Institute and State University	2
North Carolina State University	1
West Virginia Institute of Technology	1
University of Illinois	1
Renselaer Polytechnic Institute	1
University of Evansville	1
University of Minnesota	1
	18

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Table 3

Average Monthly Salaries

Categories	America - U.S. $			Canada - CA. $		
	1986	1987	1988	1986	1987	1988
College Graduate						
Overall Average	2223	2231	2487	2447	2387	—
Bachelor						
w/Prior Experience	2297	—	2713	2387	2387	—
w/No Experience	2220	2231	2400	2573	2387	—
Civil Engineering	2137	2100	2447	3153	—	—
Electrical Engineer	2380	2301	2500	—	2387	—
Co-op Students						
Overall Average			1701			
Civil Engineering			1662			
Electrical Engineering			1795			
Summer Students						
Overall Average			1569			
Civil Engineering			1569			
Electrical Engineering			1569			

Table 4

Schools of Co-op and Summer Students Sponsored by Railroads During 1988

School	Number of Students	
	Co-op	Summer
University of Nebraska - Omaha	1	
University of Nebraska - Lincoln	2	
State University of New York - Buffalo		1
State University of New York - Stony Brook		1
Morgan State University		1
San Jose State University	1	
California State University - Sacramento	1	
Oregon State University	3	
California State Polytechnical University - Pomona	1	
Southern University	1	
University of California - Berkeley	1	
University of Arizona	1	
Georgia Institute of Technology	2	
University of Alabama	1	
Clemson University	1	
North Dakota State University	7	
Montana State University	1	
University of Minnesota	1	
University of Washington	1	
Total	26	3

RECENT NORTH AMERICAN EXPERIENCE WITH SHELLING IN RAILROAD RAILS (Part I Observations)

By: R. K. Steele*

1.0 INTRODUCTION

By the mid 1940's the North American railroads had become sufficiently concerned about shelling in railroad rails to initiate studies directed at understanding the nature of shelling. The review has been condensed from AAR Report R-699 having the same title; it will treat the shell and with it, the detail fracture (TD), for which the shell is sometimes the precursor. We shall trace the progression of these studies and of the work which has followed—some of which has repeated and amplified the earlier work. Although the focus of this review will be on the North American experience, some citation of overseas experience will be made where similarity exists. Observations of shell/TD behavior and the environment in which they form and grow will be described in Part I of the review. Analytical treatments which have been developed to explain and predict shell behavior and their appropriateness will be examined in the light of actual shell behavior in Part II which is planned to be published in December A.R.E.A. Bulletin No. 723.

2.0 REVIEW

In the period from 1942 to 1948, studies (1-6) of shelly rail by Cramer concluded that there were two distinct types of shelling. One type started "inside the rail head as horizontal cracks; these horizontal cracks could turn to transverse detail fractures. The second type was described as starting "at or near the gage corner of the rail as head checks and flaking and gradually working into the rail head." The first type frequently could be recognized from a dark spot on the rail surface (gage corner) which became visible when the shell broke thru the side of the rail. The second type was not easily recognizable by visual inspection.

In 1948 Blank and Manning (7) reported on the progress of a laboratory examination of 69 shelly rails from seven different railroads. They classified the shells into two categories somewhat different from the categories described by Cramer. The categories were "(1) fractures having a vertical and a horizontal component, and (2) fractures having only a horizontal component." In the first category, the horizontal shell was 1/4 in. to 1/2 in. long connecting two transverse cracks, one of which turned down at one end and the other of which turned up at the other end of the shell. Only one of 16 rails contained an "abnormally large" inclusion (complex oxide containing iron, manganese, and silicon) which was associated with the initiation of the defect. The remaining 15 specimens contained "relatively small oxide and sulfide inclusions similar to those found in other areas . . ." The second type was termed shelly fractures. From the orientation of growth lines, the authors judged that the origins were at the gage side of the rail directly beneath a metal flow lip, the formation of which was believed to have occurred before the shell formed. They speculated that steels with greater work hardening capability would be less susceptible to this type of shelling.

One of the intents of the Blank and Manning work was to determine whether discernible differences existed between rails which shelled and those that did not. As a first step in this direction, they compared the work hardening characteristics of three pairs of standard carbon rails and three 3% chromium rails. One rail of each standard carbon pair had exhibited shelling while an adjacent rail from the same heat had not. They subjected specimens taken from each rail to cold rolling and then determined tensile and impact properties. They concluded that the work hardening behavior of the shelled rails was not different from that of the unshelled rails. They noted, in passing, that the unshelled rails were somewhat stronger and harder. Although the results for the 3% chromium rails were not reported at the time of the report, they commented that the shelled 3% chrome rails were appreciably stronger than the unshelled standard carbon rails (the curvatures from which the rails were taken were only slightly different). They pointed to the influence of microstructure; the standard carbon rails were

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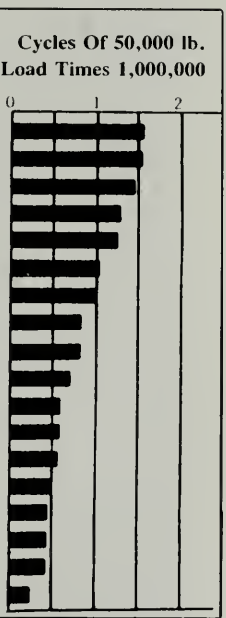
pearlitic while the shelled 3% chromium rails were of a coarse bainitic microstructure. The unshelled 3% chromium rail had a much finer bainitic microstructure. For each standard carbon pair, the unshelled rail had a somewhat higher carbon content than did the shelled rail.

In the decade of the 1940's, the rolling load tester had been perfected to the extent that it provided a means for comparing the shelling resistance of different rail steels. Crammer summarized the results available in 1948(7) which are reproduced here in Exhibit 1. Control cooled molybdenum containing 3% chromium, intermediate manganese with molybdenum, and heat treated intermediate manganese rail compositions exhibited substantially better performance than did the standard carbon rail.

In 1950 Cramer again reported (8) on shelled rails taken from revenue service as well as on the results of additional rolling load tests. The shelled rails fell into two categories, somewhat similar to those reported by Blank and Manning—namely those with origins along the gage corner of the rail and those with truly internal initiation. The two types are illustrated in Exhibit 2. Although not mentioned in the report, note that a deeper internally initated shell clearly exhibited a small inclined step at a point along its breadth. Note also that the shell occurred approximately at a depth just beneath the plastically deformed region. The near-surface cracks shown on the gage corner (Exhibit 2) today would be classified as spalls or shelly spots.(a)

Rail No.	Kind of Steel	Brinell No.
82 3-H-1	Laboratory Heat Treated Int. MN. Rail	385
5 Ui	Used 3% Chrome Rail	360
2 CC	Molybdenum Billet As Rolled	280
1091-2	Used Intermediate MN.-MO Rail	263
1091-1	Used Intermediate MN.-MO Rail	267
820	Standard Carbon Rail	260
4CC	Molybdenum Billet As Rolled	271
1094-1	1% Copper Rail	272
1094-2	1% Copper Rail	274
OX. 8	Oxweld Flame-Hardened Carbon Rail	360
OX. 6	Oxweld Flame-Hardened Carbon Rail	355
OX. 9	Oxweld Flame-Hardened Carbon Rail	365
OX. 3	Oxweld Flame-Hardened Carbon Rail	370
OX. 7	Oxweld Flame-Hardened Carbon Rail	360
*3AC	Molybdenum Billet As Rolled	292
1092	Oxweld Flame-Hardened Carbon Rail	360
1093	Oxweld Flame-Hardened Carbon Rail	365
*IAC	Molybdenum Billet As Rolled	270

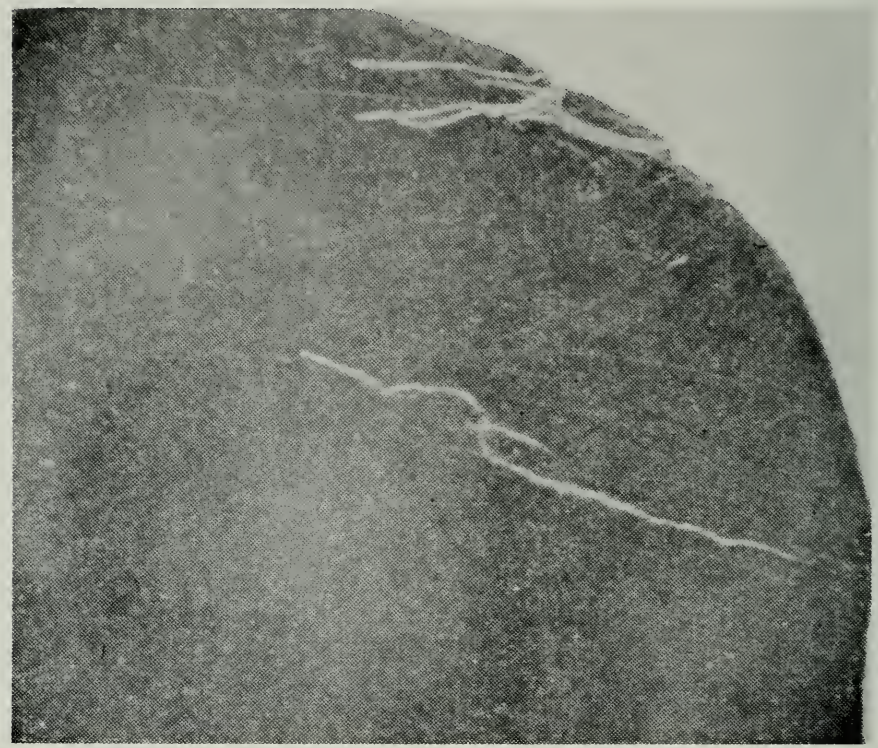


*Air Cooled Billet Containing Shatter Cracks

Exhibit 1. Rolling Load Tests Made in Cradle Type Machine.

Source:Reference 6

(a) Historically the use of the word shelling has been somewhat loose; both defects initiating internally and those initiating at or near the gage corner surface have been termed shells. Because the mechanism of initiation may be very different for each (one occurs in virtually undeformed metal and the other initiates in heavily deformed metal), the tendency today is to call the deeper defect a shell and the shallow one a spall.



Darkfield illumination shows polishing scratches as thin white lines. The cracks are outlined with Magniflux powder. The flaking cracks on the gage corner would develop into shallow shelling. The lower internal crack represents deeper shelling.

Exhibit 2. Cross Section of Rail with Two Types of Shelling Cracks.
Source: Reference 8

Among the rolling load tests was a chromium, silicon, vanadium rail which developed a shell prematurely. A streak on the surface of the shell was found subsequently to be a cluster of non-metallic inclusions; the premature shell initiation was attributed to the presence of the cluster. Also in this test series were early oil quenched rails of standard carbon content. Their shelling resistance was found to be several times greater than that of standard carbon rail.

In the same bulletin, Campbell, McIntire, and Manning (9) reported upon a detailed characterization of 54 detail fractures. More than 70% of the specimen rails were taken from the high side of curves. They grouped the rails as follows:

(a) horizontal component less than 1/8 in.
(b) "S" type having a short horizontal component with oppositely directed transverse components at either end.
(c) "L" type having a transverse component at one end only.
(d) "T" type wherein the shell component extended to either side of the vertical component.

For comparison, 26 rails removed because of engine burns, but containing no indication of shelling, were examined.

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Although some shells and detail fractures exhibited an oxidized appearance, many of those (but not all) were judged to have had internal origins. Shell components of the cracks typically were 3/16 in. to 1/4 in. below the surface. Longitudinal lines were observed frequently along the horizontal component of the cracks.

The authors noted that a section taken across one of the horizontal lines revealed that the "line" was a short vertical step in the shell. Apparently, these horizontal lines are the same as the streak observed by Cramer. The authors noted that "only 6 specimens out of 46 examined (rolled since 1 January 1940) had more than the normal distribution of inclusions in this area (of shell depth), or had a few inclusions of abnormal size." Only "two of the specimens examined had about one percent or more of ferrite at the grain boundaries of the pearlite; the balance of the specimens were entirely pearlitic." Even though horizontal lines sometimes were observed on the shell surfaces, apparently sections were not taken thru the shells themselves in order to establish whether the horizontal lines were clusters of inclusions.

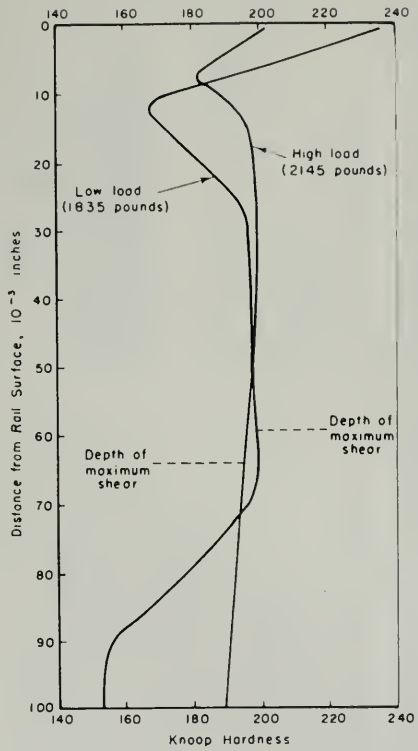
Again the issue of whether detail fracture rails were discernibly different from non defect-containing rails arose. The authors noted that although "some variation in distribution exists for some of the properties of the random rails and detail fracture rails," they "do not provide a lead to the cause of the fractures and probably are not statistically significant." Regrettably the Campbell, McIntire, and Manning investigation did not report upon the position of the shells relative to the deformation of the surface. Also, the character of the non-metallic inclusions which were sometimes found in the vicinity of the shell origins was not reported.

The issue of the effect of surface deformation on the initiation of shells was addressed by Hyler and Grover in the early 1950's (10). Working with mild steel and silver chloride they undertook rolling load studies to relate the occurrence of shell defects to the pattern of plastic deformation. The mild steel was selected (instead of rail steel) because it was believed it would exhibit deformation bands when deformed, strain aged, and etched with Fry's reagent. Silver chloride was selected for test because it exhibits mechanical behavior much like that of many metals and exhibits slip-line formation during deformation. In the tests of mild steel, maximum shearing stresses of 25 to 27 ksi were achieved. In addition, two sections of conventional rail (one as-rolled and the other heat treated) which had been subjected to rolling load test also were examined for comparison. The hardness gradients beneath the running surface are reproduced in Exhibit 3. In most cases, the maximum hardness developed at about the same depths that the maximum shear stresses were calculated to occur. And that depth did not change with the number of cycles of loading. Most significantly though, when a shell crack did develop, it formed at approximately twice the depth of the maximum hardness. A similar pattern of behavior was observed for the two standard rail specimens except that the depth at which the shell initiated was approximately 3 times greater than that at which the maximum hardness occurred (Exhibit 4) in as-rolled rail and perhaps 6 times deeper for heat treated rail.

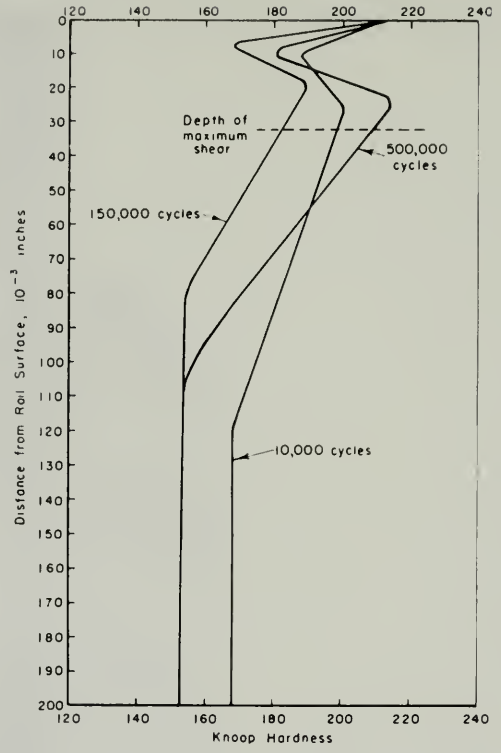
Their observations of deformation in silver chloride suggested that deformation could be progressive, extending "closer to the surface and deeper into the rail as load repetitions increased." Little change occurred after 150,000 cycles except for the shift of deformation to greater depths in concert with surface erosion.

The fact that shell initiation repeatedly had been observed in numerous investigations to occur well beneath the depth of maximum contact shear stress should have suggested that the direct action of the contact stresses alone was not the sole cause of shelling. Even though Hyler and Grover noted the depth discrepancy between the shell initiation and the maximum shear stress occurrence, they make no suggestion as to what other factors might influence shell initation.

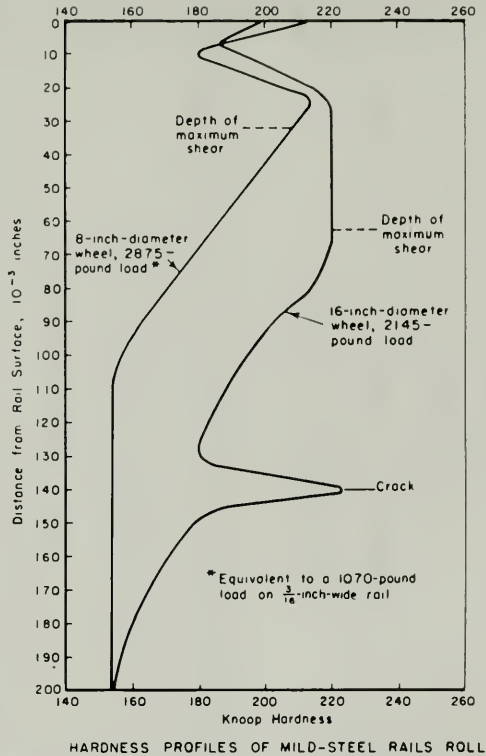
The recognition that plastic flow plays a major role in shell initiation prompted the studies undertaken by Code (11). The intent of his work "was to determine the amount of plastic flow which takes place in the gage corner of a rail . . . under conditions which produce shelling." By inserting


Distance from Rail Surface, 10^{-3} inches
Knoop Hardness
High load (2145 pounds)
Low load (1835 pounds)
Depth of maximum shear
Depth of maximum shear

HARDNESS PROFILES OF MILD-STEEL RAILS ROLLED WITH 16-INCH-DIAMETER WHEEL FOR 150,000 CYCLES



HARDNESS PROFILES OF MILD STEEL RAILS ROLLED WITH 8-INCH-DIAMETER WHEEL WITH LOAD OF 2875 POUNDS


Distance from Rail Surface, 10^{-3} inches
Knoop Hardness
Depth of maximum shear
8-inch-diameter wheel, 2875-pound load*
Depth of maximum shear
16-inch-diameter wheel, 2145-pound load
Crack
*Equivalent to a 1070-pound load on $\frac{3}{16}$-inch-wide rail

HARDNESS PROFILES OF MILD-STEEL RAILS ROLLED WITH VARIOUS DIAMETER WHEELS FOR 500,000 CYCLES

Exhibit 3. Hardness Profiles of Laboratory Test Specimens
Source: Reference 11

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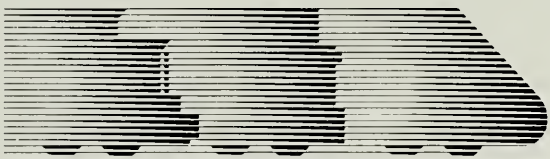

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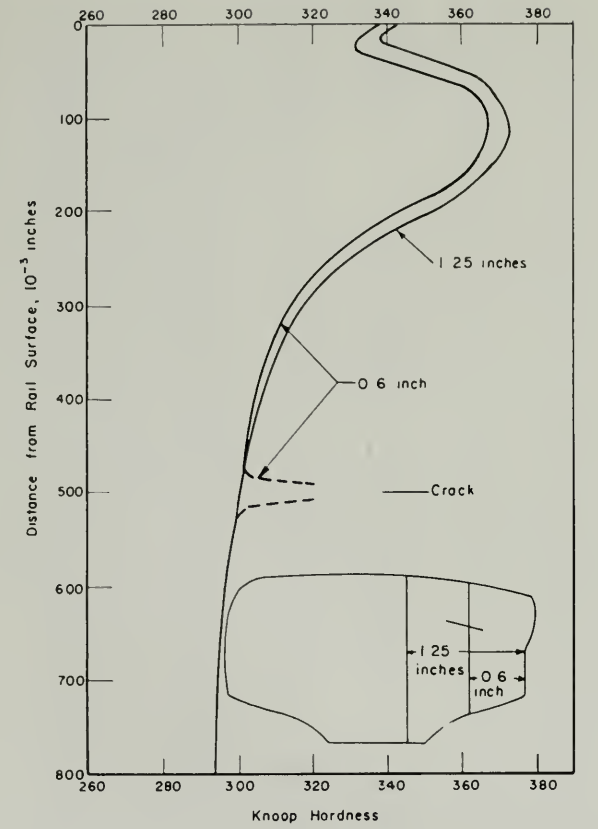
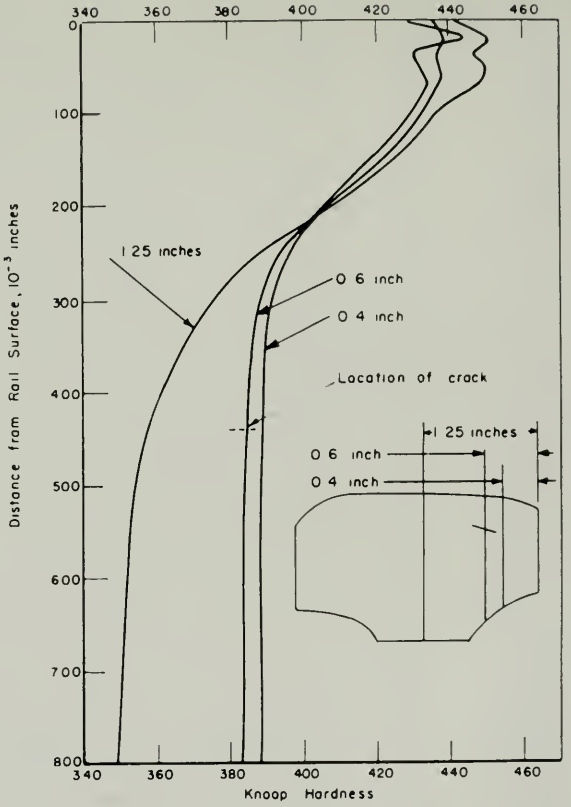

260 280 300 320 340 360 380
0 100 200 300 400 500 600 700 800
Distance from Rail Surface, 10^{-3} inches
1 25 inches
0 6 inch
Crack
1 25 inches
0 6 inch
Knoop Hardness

HARDNESS PROFILE OF NON-HEAT—TREATED STANDARD RAIL SPECIMEN NO 828


340 360 380 400 420 440 460
0 100 200 300 400 500 600 700 800
Distance from Rail Surface, 10^{-3} inches
1 25 inches
0 6 inch
0 4 inch
Location of crack
1 25 inches
0 6 inch
0 4 inch
Knoop Hardness

HARDNESS PROFILE OF HEAT-TREATED STANDARD RAIL SPECIMEN NO 1139

Exhibit 4. Hardness Profiles of Rails taken from Revenue Service
Source: Reference 11

brass pins at different angles into the gage corner side of the rail and then observing the deformation of these pins (by metallographic sectioning) after a period of traffic, he was able to conclude that:

(a) "the principle flow is parallel to the tread of the wheel."

and

(b) "the maximum depth of visible movement is 0.20 to 0.40 in., so that in general it may be said that flow extends 1/4 in. to 3/8 in. below the surface of the rail head. It will be noted that this is the depth at which the metal separation in a shelly spot is generally observed."

Code summarized many of the observations related to the shelling problem in 1960 (12). He reminded his audience of the effect of curvature and lubrication to accentuate shelling and of greater rail strength to reduce shelling. The issue of ductility exhaustion due to excessive plastic flow was raised (even though the shell develops below or toward the bottom of the region of plastic deformation). A mention is made of a study of residual stresses which "produced nothing of practical value." Yet the possible contribution of residual stresses will arise again in more recent work. Code proposed that to avoid the damage inflicted by plastic flow, the wheel load must be below 600 lb. per inch of wheel diameter. To contain the problem at a familiar, if not entirely acceptable level, he urged that wheel load should not exceed 800 lbs. per inch at wheel diameter. Today's expected North American maximum, but hopefully not too frequent, wheel loads near 39 kips would necessitate very large wheels indeed (49 in. dia.) were this criteria adhered to.

In 1959 Kalashnikov (13) described the occurrence of shelling type defects in "a particularly high proportion of rail . . . (about 50%) . . . from the Donets line" in the USSR. He reported that the "cracks form at the boundary between cold worked layers and metal which has not been cold worked and these cracks become deeper under continued alternating loading in service conditions." In a passing comment about reduction of wear, he cites work by Koziichuk (14) recommending that "the form of contact between tyres and the rails should be equalized." No mention was made of the possible beneficial effect on contact stress.

Shkol'nik (15) noted that in the Soviet experience "non-metallic inclusions and, in particular, their agglomeration" facilitated the development of deep horizontal layers in the heads of rails and their subsequent surface stratification resulting in the formation of chipping." His observations did *not* confirm other findings that had indicated that (shell) cracks developed on the gage side of the rail head directly under the metal flow lip and that crack initiation and growth preceded the metal "overflow." Instead he found that metal flow to the gage side occurred before an external manifestation of a surface crack was evident. He indicated that residual stresses could contribute to the conditions favorable for plastic flow. But it is not clear that he believed that surface plastic deformation could alter the internal residual stress state of the rail head. He made a further interesting observation that if "chipping" (presumably break off of the material above the shell) occurred soon after cold working (stratification) then a transverse defect (detail fracture) would *not* form from the shell. This observation should be remembered when we reach the analysis of shell turning behavior.

In 1965, Ito and Kurihara (16) reported upon a rapid increase in the number of shelling rails on some of the rail lines in the vicinity of Tokyo and in the Shikoku Regional Office. The rapid rise began in the 1962-63 period. The defects were remarkably similar to the shell/detail fracture found in North American service under much heavier axle loads. The shell depth was 3 to 7 mm with widths of 5 to 22 mm. The lengths ranged from 20 mm to several hundreds of millimeters. Again most commonly the defect developed under the gage corner on the high rails of curves. The onset of the problem seemed associated with the presence of diesel rail cars initially introduced in 1957 and 1958 and the presence of improved lubrication achieved initially in 1962 by the introduction of on-board lubricators. Shells began to appear after approximately 100 MGT of service exposure, 20 MGT of which was lubricated in rails containing 0.62-0.69 w/o C, 0.71-0.78 w/o Mn, and 0.09 to 0.18 w/o Si. The shells appeared at a

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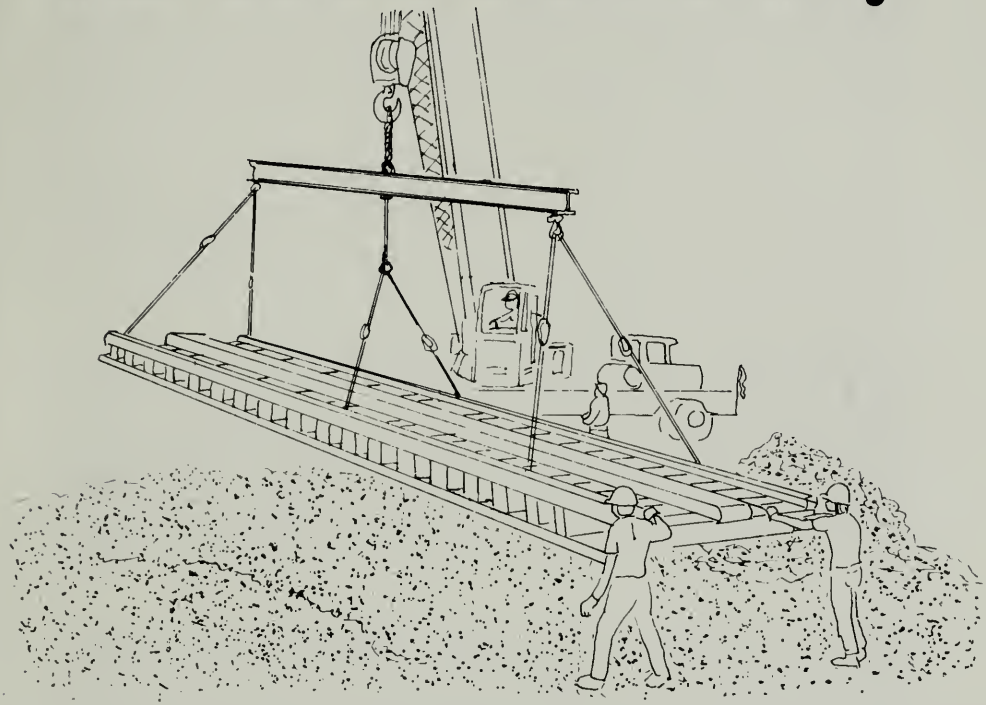

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depth well beneath the depth of maximum work hardening again near the transition from plastically deformed to undeformed base metal. By far, the greatest number of shells occurred in the inclusion-prone top rail from an ingot and the least occurred in the bottom rail of the ingot.

Eisenmann (17) described the development of shell defects under German operating conditions in 1969. He noted that failure started "at a depth of 0.2 to 0.3 inches where stress assumes its extreme value" and that occurrence at that depth could "be only explained by assuming a compressive stress uniformly distributed over the contact surface. The assumption of an elliptical distribution according to the theory of Hertz leads to depth figures between 0.08 to 0.12 inches, which obviously do not agree with the experience." However, he reported photoelastic test results with an epoxy resin rail that suggested the occurrence of maximum shear stresses at a depth of 2-1/2 mm (0.1 inch) in good agreement with Hertzian theoretical calculations. The extent and distribution of the plastic deformation which resulted from the contact shear stresses apparently were not determined. If German experience were similar to that of North America, the USSR, and Japan, the shell would have formed toward the bottom side of the zone of plastic deformation the maximum extent of which would have occurred in the proximity of the region of maximum calculated shear stress.

Examinations of North American shelled rails by Wisnowski (18, 19) in 1967 revealed the presence of a white etching layer on the surfaces of the internally initiated shells. At first the presence of white etching layer was considered "to be foreign material attributable to manufacturing practice." Similar white etching layer was reported by Stafleva et al (20) in cyclically compressed rail steel specimens. Henry (21) proposed that the white etching layer is a highly plastically deformed region in which iron carbides have been re-dissolved by a precipitate reversion mechanism similar to that proposed by McEvily and Boettner (22) for torsion fatigued aluminum alloys. In as much as the white etching layer had *not* been found in the solid base metal but only on the shell surfaces, one might be led to believe that the severe deformation conditions necessary for formation of the layer are achieved only in very local regions on the free surface of the shell.

Gervais and McQueen (25) undertook to examine the accumulation of fatigue damage which could lead to the formation of shells. They indented a plate of Armco iron by oscillatory Hertzian loading. The load selected was such that the maximum contact pressure was 100,000 psi (689 MPa) thereby achieving a ratio of maximum contact pressure to shear yield point of 4.93. This ratio was stated to be very similar to that achieved by worn wheels on new rail on the Quebec North Shore and Labrador Railway. The maximum shear was calculated to occur at a depth of about 0.5 mm. The extent and magnitude of plastic deformation were judged from microhardness maps made upon sections taken across the contact after 0.5×10^6 cycles and 10×10^6 cycles. At 0.5×10^6 cycles, the very near surface hardness directly under the contact region had hardened to a maximum in the range of 130 to 140 DPH with work hardening gradually diminishing to a depth of 0.02 in. Beneath this point, to a depth of about 0.06 to 0.07 in., the specimen material *had become softer than the original undeformed material* (94-104 D.P.H. compared to 109.2 ± 3.8 D.P.H.). However, by 10×10^6 cycles, *a portion of the material directly under the contact which had exhibited work hardening at 0.5×10^6 cycles had become softer than the original undeformed material;* and the previously 'work softened' material at 0.02 to 0.07 in. depth directly beneath the contact had become harder than the undeformed material.

Transmission electron micrographs were made at a number of depths beneath the contact at intervals during the test. They revealed that at 0.5×10^6 cycles, the work hardened material near the surface had developed "cells with thick, dense, ragged walls." At the depths at which maximum shear stresses were calculated to occur (0.015 to 0.020 in.), sharply defined elongated cells with dislocation entanglements and loops were observed. In the deeper softened region, the dislocation density was only slightly above that of the recrystallized material. By 10×10^6 cycles, the region below the contact, where softening had been observed, exhibited a much lower dislocation density than it had earlier to the extent that the cell structure had disappeared. This reversal of structure was attributed to a lowering of contact stress resulting from a small enlargement of the contact width. The fact that cracks did not develop in the fatigue damaged region beneath the contact was attributed in part to reduction in stress and the reversion of structure.

But service induced shells might be expected to initiate toward the lower boundary of the work hardened region, so perhaps other reasons for the failure to generate a fatigue crack might be possible. One of these would be the absence of fully reversed shear that would occur upon rolling but not upon repeated indentation. Nevertheless the work shows that the depth of work hardening can become (over a period of cyclic exposure), several times greater than the depth at which the maximum shear stresses would be calculated to occur. Regrettably, the small scale dislocation structure was not examined at greater depths at the longest cyclic exposure. Also, the occurrence of cycle softening should be noted.

In 1974, Ravitskaya (26) reviewed the Soviet experience with shell initiation and growth. In particular, she focused on the occurrence of and contribution of white etching layer in the process. She observed again, as others had before, that the fatigue damage occurred at a depth of 4 to 12 mm which was well beneath the region of strain hardening (3 to 3.5 mm deep). The origin of the shell was a "nucleating strip," i.e., a step in the surface of the shell, which was 0.04 to 0.30 mm high. Electron fractographic examination revealed the presence of white etching layer on the shell surface as reported previously by Wisnowski (18, 19). Contrary to the proposal of Henry (21), x-ray diffraction studies were reported to reveal the presence of austenite lines and "considerably widening of the phase lines." These observations led Ravitskaya to conclude that the white etching layer consisted of austenite and martensite. She noted that the line "changes were not observed out side the interface region or on the transverse fatigue crack surface." The white etching layer and associated bending and partial dissolution of cementite lamellas were attributed to the occurrence of wave propagation from impact loads acting at a depth (about one-half of the contact length) where considerable "tangent" stressess occurred. No mention was made of observation of any nonmetallic inclusions in the vicinity of the "nucleating strip." Nor was there mention of any residual stress state that may have existed in the rail. Presumably, if the presence of white etching layer was essential to the initiation of the shell and its formation were the result of wheel impacts, then laboratory rolling load tests (free of impact loads) should not initiate shells; yet, of course, they do!

The 1976 ASTM symposium "Rail Steels—Developments, Processing and Use," (27) presented a number of relevant papers. In a discussion to the paper by Marich and Curcio, Koerfer (28) described shelling that is similar in appearance to that occurring in North America. The depth of occurrence was 6 to 10 mm beneath the running surface, generally toward the lower side of the work hardened region. He commented that "numerous examinations have proved that often shelling is *not* the result of internal defects in the material . . ." and ascribed initiation to " . . . the inability of the material to withstand the applied loads." However, material defects were described as "large slag inclusions, clinker, or poor material purity . . ." Evidence of deformation and the presence of hollow cavities in the vicinity of the shell were portrayed.

Sonon et al (29) reported on the metallurgical analysis "of 33 carbon steel rail samples selected from service, 31 of which had service-developed defects." This work was undertaken as part of a joint effort by the American Railway Engineering Association (AREA), the Association of American Railroads (AAR), and the American Iron and Steel Institute (AISI). One of its purposes was to examine yet again the issue of whether defect-initiating rails are different from those that do not initiate defects. Sonon et al concluded that (a) "all the service-developed defects (most of which were shells or detail fractures from shells) in the rail samples examined were fatigue cracks," (b) "all of these fatigue cracks initiated at sharp internal notches such as inclusions," and (c) "a correlation between the metallurgical properties of the rail samples and defect formation could *not* be made." Fractographic examination had revealed that the fatigue cracks had "initiated at discontinuous clusters of calcium-aluminate inclusions that formed stringers which were as small as 1 mil (0.001 in.) wide and had length-to-width ratios much greater than 10. In addition, "several fatigue cracks (compound fissures) also initiated at what appeared to be micro voids." Sulphides were observed not to act as initiation sites. Although hardness traverses (Rockwell C) were made from the gage corner inward, the location of the defects relative to the surface was not reported. Most of the defects occurred in rail laid in tangent track with only a few defects appearing in rail laid in curves. As a result, the average service exposure for the samples was 500 MGT

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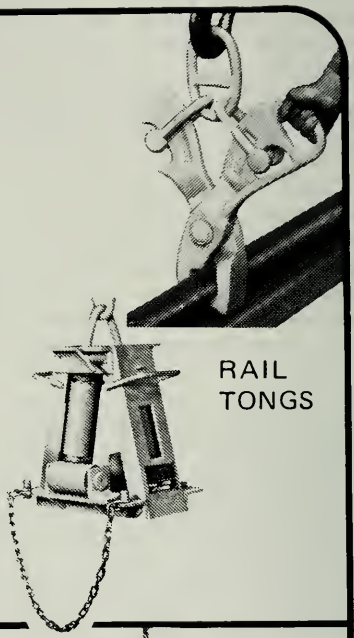

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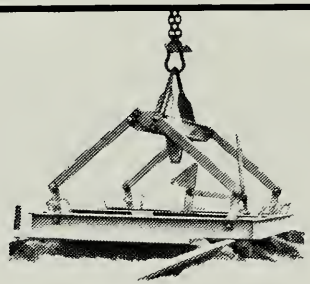

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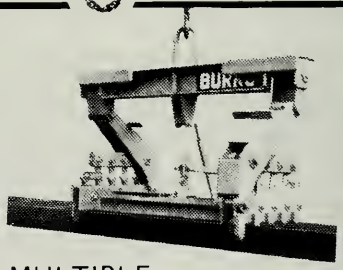

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with most lives falling in the range 363 to 545 MGT. The selection of samples predominantly from tangent track resulted from the absence of samples having shorter lives in traditionally troublesome track locations. Because of the long average life of the samples examined, the authors suggested that a study of defects that form early in the rail life would be of greater importance and interest with respect to the effect of steel quality and properties. This study revealed very much the same features that Blank and Manning (7) had reported 28 years earlier.

A companion paper by Barsom and Imhof (30) undertook to determine fatigue and fracture characteristics of some of the AREA/AAR/AISI samples. In considering the initiation of shells from nonmetallic inclusions, they concluded that "elimination of these discontinuities by changes in steel making or by nondestructive inspection is technologically and economically infeasible." Furthermore, "a significant increase in the fracture toughness of the rails would result in a negligible increase in the useful fatigue life of the rail." However, the authors did note that "fatigue crack initiation, fatigue crack propagation, and final fracture are strongly dependent on the magnitude and distribution of the *residual stresses* and the applied cyclic stresses." Regrettably, though, no calculations were made to illustrate the quantitative nature of that dependence.

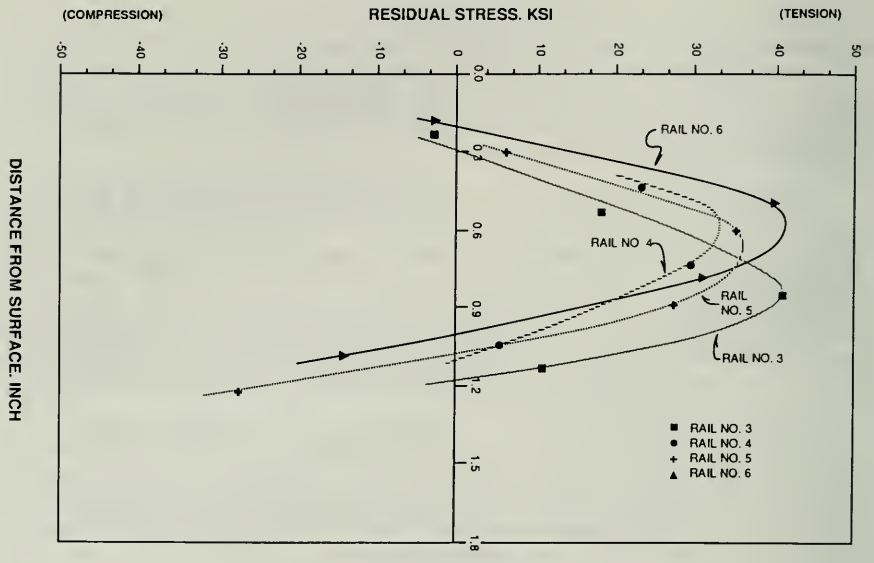
An exhaustive examination of shell/TD behavior in Australia under ore-hauling conditions was presented by Marich et al (31). Their work pointed strongly to the presence of exogenous complex silicate inclusions as the predominant agent for nucleation of the shell crack. The initiation occurred "at a step 0.1 to 0.2 mm in height. Elongated inclusions at least 400 m in length were observed embedded along the surface of the step . . ." They confirmed the depth of subsurface initiation as 5 to 7 mm beneath the gage corner - perhaps slightly less "in the high rails from 2° to 3° curves." They attributed the difference to the greater wear occurring on curves. In line with the previous observations of Wisnowski, Henry, Stefleva, and Ravitskaya, they observed a white etching layer on crack surfaces. However unlike some previous work, they observed its occurrence on the surface of transverse cracks. They concluded that the white etching layer "is due to very rapid and localized thermal cycling and deformation which result from friction at the rubbing surfaces of the shelly cracks and transverse defects in their *early* stages of growth." Issues that they did not address were (a) where the shell was located vis-a-vis the plastically deformed surface and (b) what were the service-induced residual stresses.

Although earlier work (23, 24) had made reference to the contribution of residual stresses, little was known of the level and distribution of service induced residual stresses in North American rails up until the mid 1970's. In parallel, independent studies Groom (32) and Schilling and Blake (33) reported on the results of destructive sectioning tests to determine the service induced residual stresses.

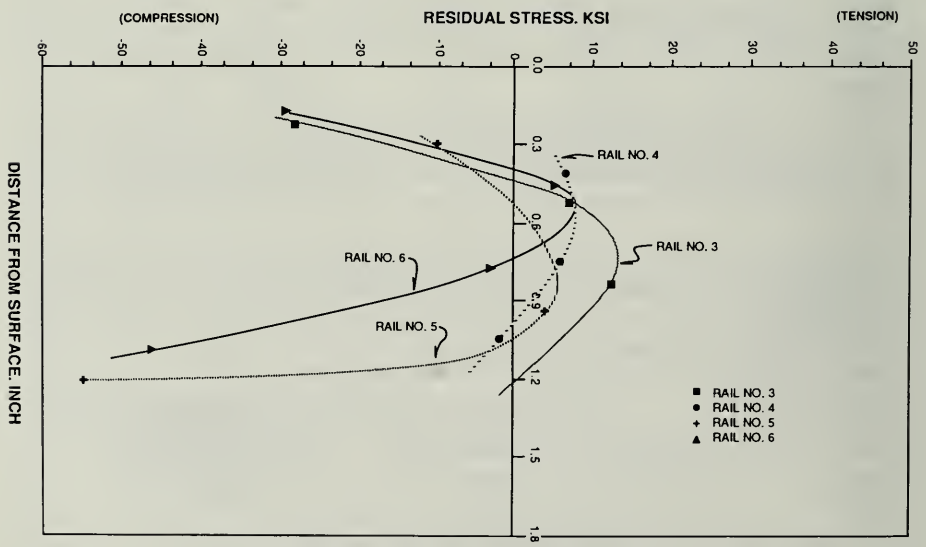
The depth distributions of the three dimensional residual stresses for a number of rails determined by Schilling and Blake 0.6 in. from the rail center are portrayed in Exhibit 5. Rails 3, 4, 5, and 6 had been exposed to approximately 150, 200, 450, and 600 MGT of traffic respectively. It is not clear that the service exposed rails had seen similar wheel loads and track conditions. But on the basis of the observed scatter in the positions of the curve, one might conclude that a traffic exposure effect is not unequivocally evident. Generally the behavior could be summarized as follows:

Stress Direction	Max Tensile Stress Level	Depth at Max Tensile Stress
Transverse	35-40 ksi	0.8-0.9 in.
Vertical	5-10 ksi	0.5-0.7 in.
Longitudinal	20 ksi	0.7 in.

Typically, for the 0.6 in. off-centerline section, the depth at which the residual stress field changed from compression to tension (with increasing depth) was about 0.3 in., very close to the expected bottom of the work hardened zone and approximately at the depth where shells were observed to occur. However, the 0.6 in. off-centerline section may not be the section upon which the shell initiation will occur.



Transverse Stress



Vertical Stress

Exhibit 5. Residual Stress Distribution on a Line 0.6″ from the Centerline in Service-Load Rails.

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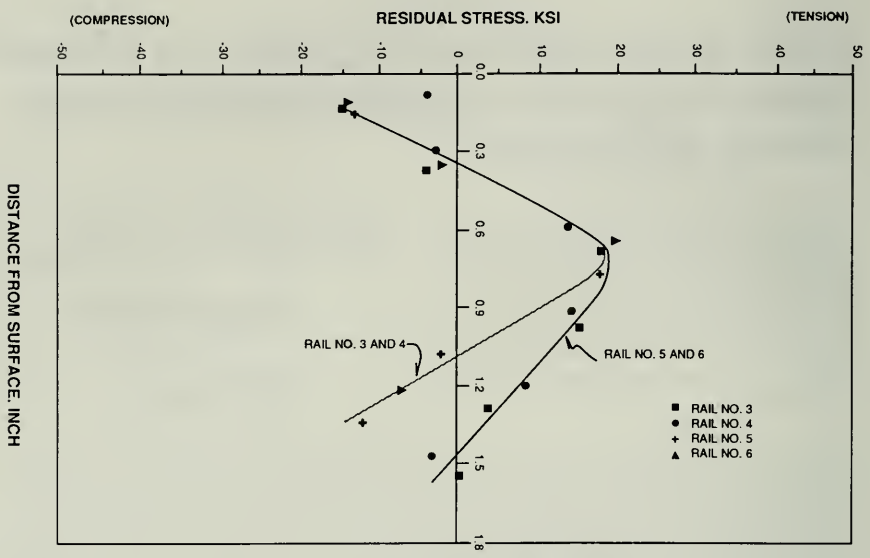
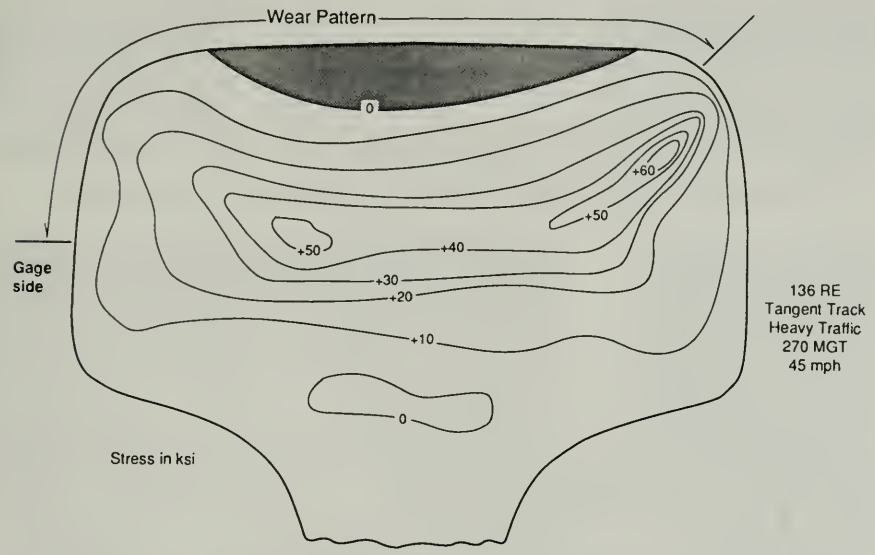


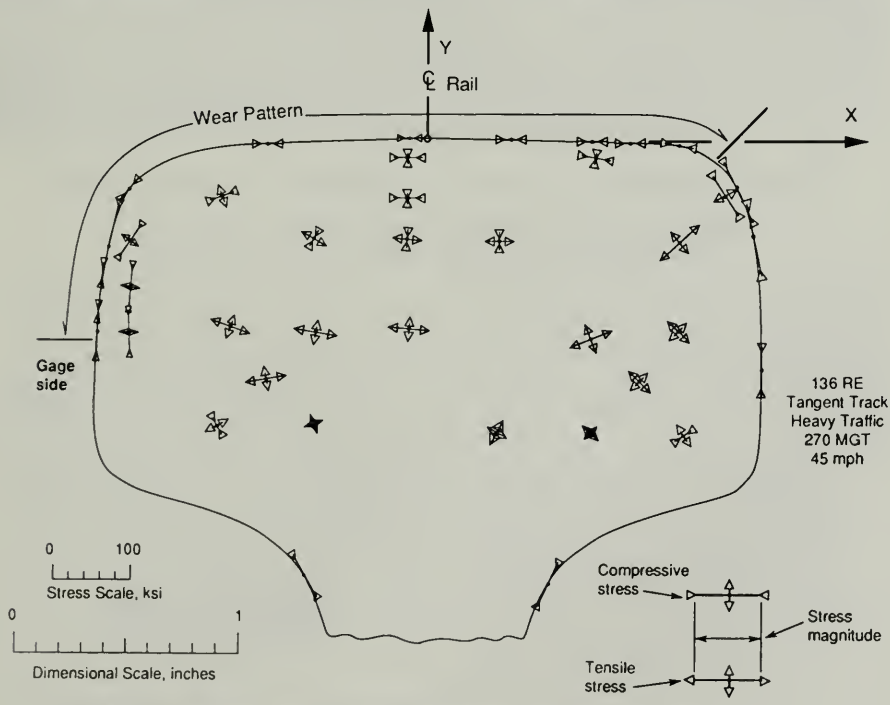
Longitudinal Stress

Exhibit 5 Continued.

The results obtained by Groom were similar in nature to those of Schilling and Blake. Typical data from this study are plotted as stress maps in Exhibit 6 for in-plane (transverse) residual stresses. As with the results of the Schilling and Blake work, the maximum residual tensile stresses occur at depths of approximately 0.8 inches. An advantage of a map type of portrayal is the ability to locate the region of maximum stress either in tension or compression. As can be seen in Exhibit 6, the region of maximum residual tension *can* occur toward the field side edge of the wheel path (wear pattern) although this has not always happened. The maximum in-plane tension can reach as high as 60 ksi. However, at the depth of 1/4 in. to 3/8 in. under the gage corner where shells can develop, the in-plane tensile residual stress reached 10 to 30 ksi with the principle stress direction approximately perpendicular to the shell plane and slightly inclined to the running surface of the rail. Although the selection of rail samples for stress determination was not ideal for determining the effects of wheel load and service exposure, the data presented in Exhibit 7, suggest that heavier wheel loads and greater service exposure do tend to increase the in-plane (transverse) residual tension stresses. However, this observation must be taken to be somewhat indefinite because the rail specimens could not be taken from a carefully controlled population having the same properties and exposed to identical loading conditions throughout service exposure. The presence of modestly high residual tensile stresses at the depth of and approximately perpendicular to the plane of the shell suggests that the service induced residual stresses could well play a part in the initiation of the shell. But, the fact that the shell does *not* initiate at the position of maximum in-plane residual tension suggests that another stress contribution must come into action as well. Perhaps that other stress contribution is the contact stress system. More will be said of this.



SPECIMEN NUMBER 2, IN-PLANE TENSILE STRESS MAGNITUDE CONTOURS



SPECIMEN NUMBER 2, IN-PLANE PRINCIPAL STRESS VECTORS (SLICE 2)

Exhibit 6. Residual Stress Maps (Specimen 2).
Source: Reference 32

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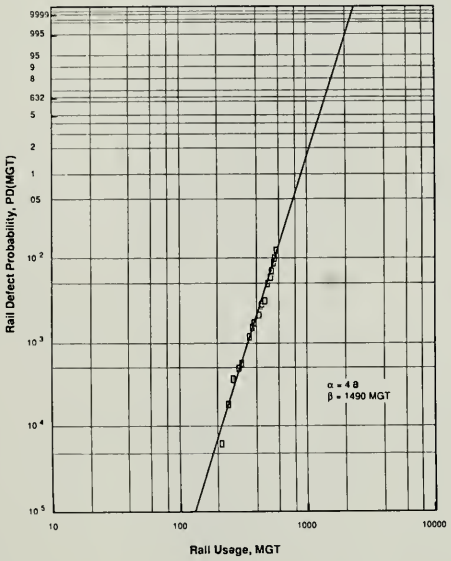
Table 1. Maximum Residual Stresses Measured on Each of the Tangent Rail Specimens

Specimen No. and Description	Maximum Residual Stress Component, ksi				
	(A) Peripheral on the (Surface, Compressive)	(B) In the Transverse Plane (Subsurface, Compressive)	(C) Axial (Compressive)	(D) In the Transverse Plane (Tensile)	(E) Axial (Tensile)
1. Heavy Traffic 83 MGT	-142.7	-49.0	-91.0	41.0	17.3
2. Heavy Traffic[a] 270 MGT	-100.7	-66.5	-63.8	64.3	22.0
5. General Traffic 100 MGT	-59.2	-34.3	-39.7	35.9	18.9
6. General Traffic 300 MGT	-40.2	-59.8	-31.6	41.3	18.4

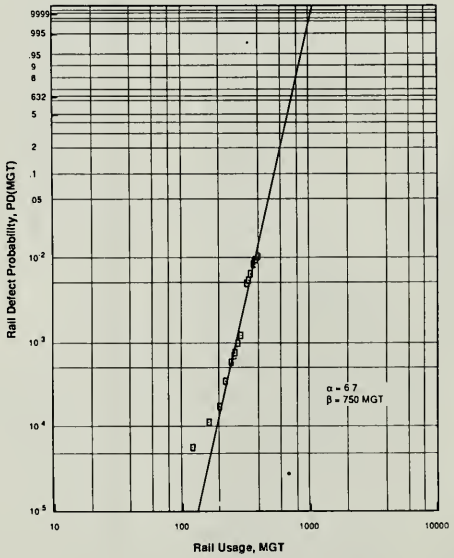
(a) The stresses shown are the maximums from either of two locations where measurements were made.

Exhibit 7. Maximum Residual Stresses Measured on Each of the Tangent Rail Specimens Source: Reference 32

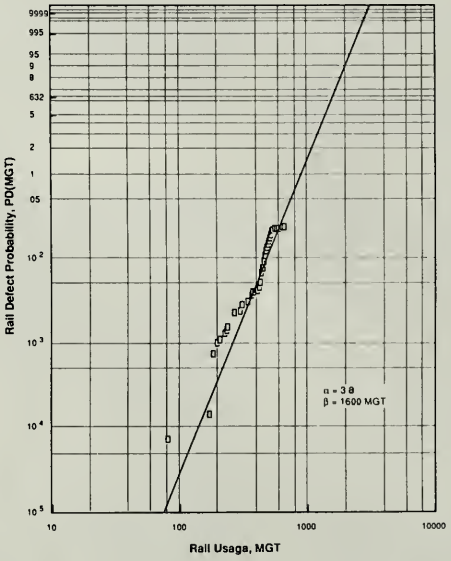
A topic which seems not to have been treated until the work of Besuner et al (34) is the service exposure-to-occurrence distribution of shells. The rail defect/failure statistics from six test sites were utilized to calculate cumulative probabilities which were then presented as Weibull plots. The six test sites were made up of four on the Union Pacific Railroad and two upon the Santa Fe. The Union Pacific sites ranged from 51 to 63 miles in length while the two Santa Fe sites were 22 and 29 miles in length. The accumulated service exposure ranged from a low 397 MGT to a high at 677 MGT. On average, 86% of the defects/failures from the Union Pacific sites were associated with shells (either shells alone or detail fractures from shells). But on the Santa Fe, the percent of defects associated with shell occurrence was much lower: 29% for one site and 45% for the other. In three of the six cases, the data yielded linear Weibull plots. But, in the other 3 cases, a tendency toward decreasing defect rate at greater service exposure was obtained. These behaviors are portrayed in Exhibit 8. Where a linear fit has been possible, the average Weibull slope was about 3.6 and the characteristic lives varied from 1250 to 2350 MGT. The important finding in this work is the realization that rail fatigue defects, most especially shells, can be characterized by relatively well-behaved life distributions such that one cannot specify a single fatigue life for rail but rather must describe the life distribution. An important question of current interest is how parameters such as wheel load, track character, rail design and metallurgical character and especially wear rate affect the life distributions.



Cumulative Probability Distribution for Detailed Fracture Shell (DFS) Type Defect at the Union Pacific #1 Site.



Cumulative Probability Distribution for Detailed Fracture Shell (DFS) Type Defect at the Union Pacific #2 Site.



Cumulative Probability Distribution for Detailed Fracture Shell (DFS) Type Defect at the Union Pacific #2A Site.

Exhibit 8. Revenue Service Defect Cumulative Probability Distributions.
Source: Reference 34

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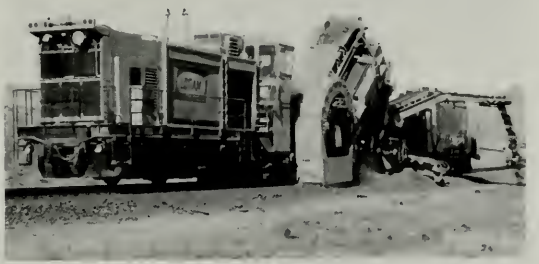
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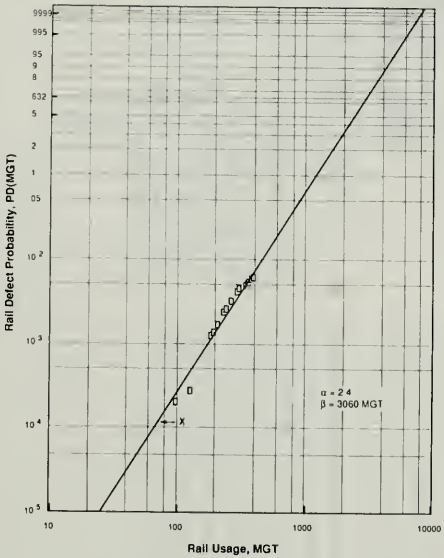
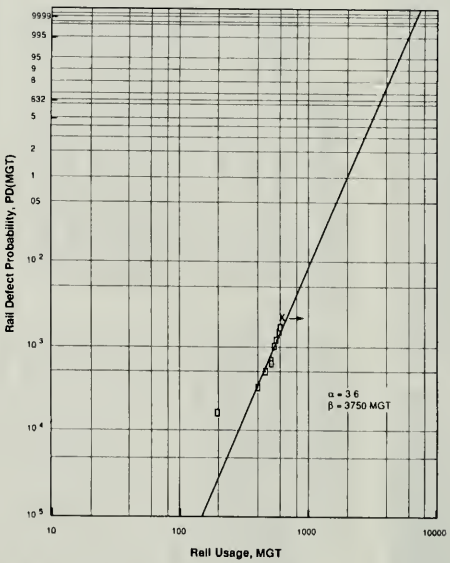
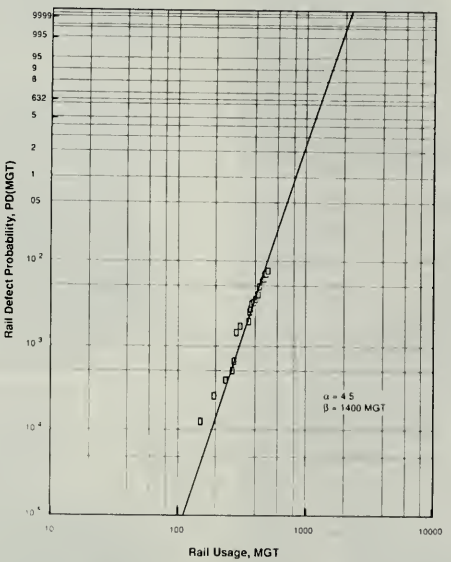


Cumulative Probability Distribution for Detailed Fracture Shell (DFS) Type Defect at the Union Pacific #3 Site.



Cumulative Probability Distribution for Detailed Fracture Shell (DFS) Type Defect at the Santa Fe #1 Site.



Cumulative Probability Distribution for Detailed Fracture Shell (DFS) Type Defect at the Santa Fe #2 Site.

Exhibit 8 Continued.
Source: Reference 34

Ghonem et al (35) reported the surface initiation of shells on the gage face of high rails from curves. Their observations bore a superficial resemblance to those of Cramer (8). They indicated the position of the shell to be at the lower boundary of the work hardened region. X-ray diffraction residual stress profiles (taken radially in uncracked metal across the mid width of the shell on a transverse plane) revealed that significant residual tension stresses (20 ksi) had acted perpendicular to the shell surface. They attributed the gage face surface initiation to the growth of intrusions formed in metal heavily deformed by flange contact. Opening (and growth) of the intrusion was hypothesized to occur under the action of large tensile stresses which can occur at the rear boundary of the gage face/flange sliding contact zone. Presumably these tension stresses are greatest when the coefficient of friction is high[a] and will be substantially smaller when the gage face is lubricated.[b] The region thru which the shell grew was cited(36) by authors to be work softened although the hardness contours presented in the paper did not show any indication of a softened region. A view of the actual shell surface had not been shown, so that the reader cannot judge the nature of the actual growth behavior of the shell and any associated traverse component.

The actual growth of the shell and its tendency to turn has been difficult, (well, at least speculative), to infer from the post-mortem examination of shell defects generated in revenue service or even in the rolling load tester. However, the reversing train operation at the Facility for Accelerated Service Testing (FAST) has afforded a unique opportunity to track the growth of the shell and its associated detail fracture and at the same time characterize the environment in which the shell has grown. The reversal of the train at approximately 1 MGT (30,000 cycles) intervals has caused periodic changes in crack trajectory such that the position of the crack can be correlated with specific periods of service exposure. A description of shell growth and its turning to a detail fracture has been given by Steele and Reiff (37). Exhibit 9 shows the appearance of both a shell and a typical detail fracture. The shell initiated internally beneath the gage corner of the rail (0.3 in. depth) at a number of small longitudinal steps (Exhibit 10). Microhardness maps made on transverse section (Exhibit 11) nearby to the shell (but not including the shell crack) suggested that the shell may have initiated in a lens of metal softer than that which surrounds it. Such lenses are detected only occasionally and then not with great certainty. Their appearance is somewhat reminiscent of the observations of Gervais and McQueen.

Although nonmetallic inclusions were found near the shell surface, neither their number nor their size were exceptional. Strangely, no indications of nonmetallic inclusions were found on the shell surface at the vicinity of the origin.

The residual stresses which had developed within the rail are shown in Exhibit 12 (38). The shell initiated in a region of only modest residual tension somewhat less than 10 ksi tension longitudinally and about 10 ksi in a direction approximately perpendicular to the running surface over the shell. The shell grew radially from its origin for at least 10 MGT before it had travelled far enough to reach the gage face of the rail. That 10 MGT (minimum) interval also corresponded to a travel of about 3/4 in. along the rail whereupon the transverse crack (detail fracture) developed. Examination of a

(a) The coefficient of friction will be highest in the unlubricated condition. But, this will encourage dry wear on the gage face which will tend to 'wear away' the intrusions thereby limiting the further growth inward of an intrusion

(b) Subsequent lubrication would reduce wear thereby prolonging the time for intrusion growth. But the presence of lubricant on the gage face would decrease tension stresses thereby decreasing tendency toward further growth. Perhaps lubricant in the intrusion would cause hydraulic action to propagate the intrusion further

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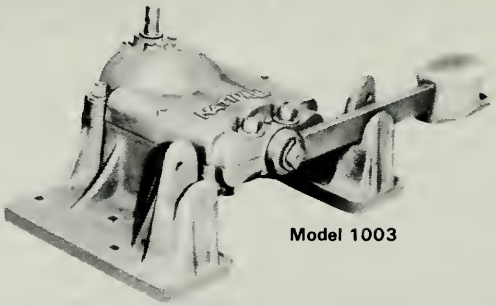

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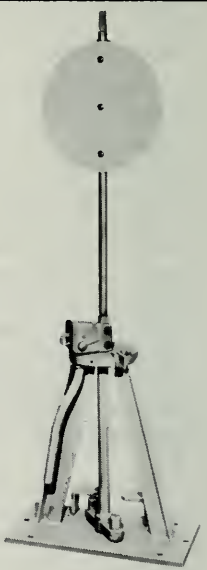

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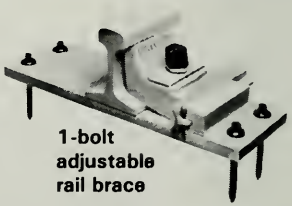

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Typical FAST detail fracture

Typical FAST shell

Exhibit 9. Views of Shell and Detail Fracture.
Source: Reference 37

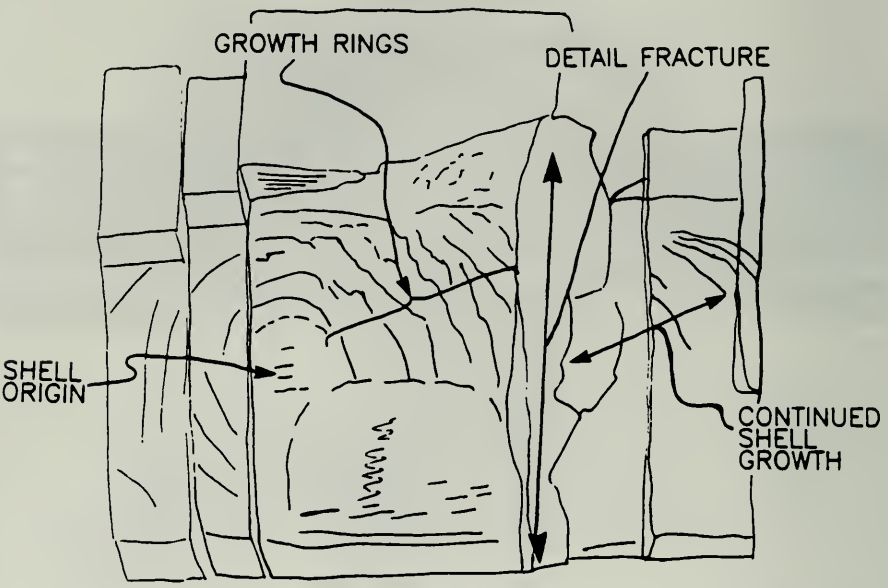

Exhibit 10. View Down from Running Surface with Metal Above Shell Removed Showing Salient Features.

Source: Reference 39

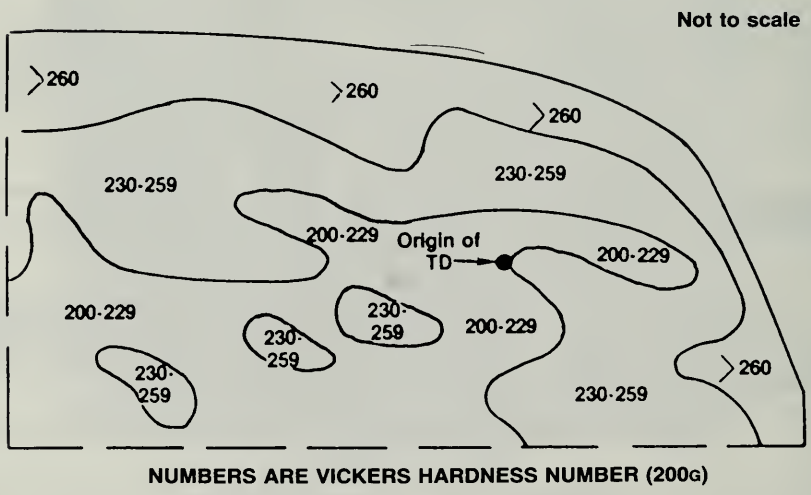

Exhibit 11. Hardness Map of TD-Containing Rail.
Source: Reference 37

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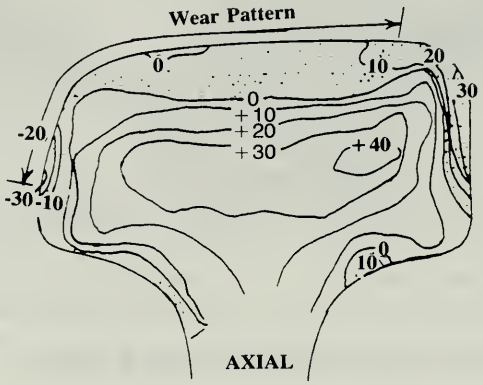
longitudinal section taken through the shell showed that many of the valleys of the shell crack wavelike pattern contained inclined cracks extending into the base metal. These cracks seem not to have progressed very far. Eventually as the shell had become longer, one of these cracks became active enough to initiate growth of the major transverse crack. Even as the transverse component grew, the shell continued on, although its tendency to exhibit wave-like peaks and valleys seem to diminish. In the light of what we believe today from the analysis of Farris et al (39), the shell appeared to reach a point of maximum turning tendency after about 3/4 in. of longitudinal growth. More will be said of that analysis in the second part of this paper.



SOURCE: REFERENCE 38

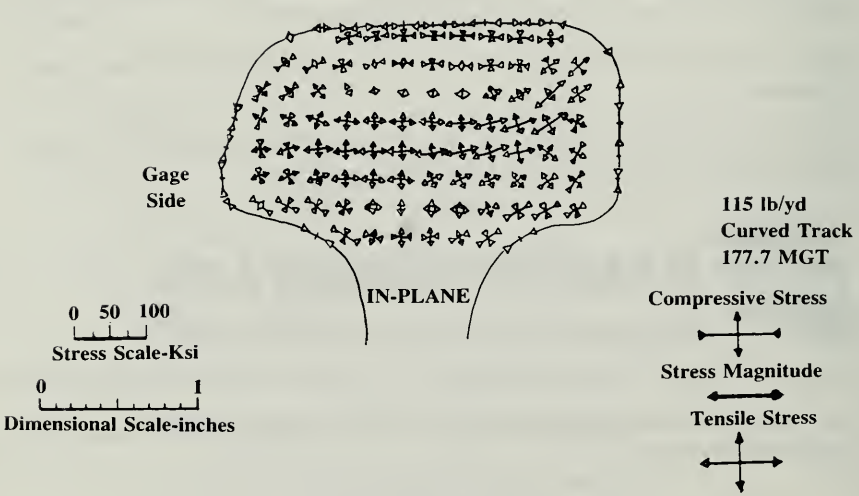


Exhibit 12. Axial and In-Plane Residual Stresses in a Service Worn Rail.

The wavelike pattern which appeared on the *transverse* portion of the crack and most especially the fact that the waves became deeper as they progress up to about one half of the distance across the head, presents a provocative question. If the transverse crack grows primarily under the influence of the service induced longitudinal residual tensile stresses *and* the active longitudinal beam bending tensile stress (which is calculated to be maximum when the wheel is about 65 in. away from the defect), what is the stress system that causes the wavelike pattern of growth on the transverse plane? Certainly contact shear stresses and beam shear could be expected to cause a rotation of the principle stresses acting on the crack when the wheel passes over the crack. But, when the transverse crack clearly would be open (i.e. growing) these shear stresses would be calculated to be acting at a point about 65″ away from the defect and would not be expected to have any direct effect.

The unique crack growth tracking ability that FAST testing has afforded has led to a crack growth experiment known as FASTKRAX. In this experiment, rails containing detail fractures (and shell/detail fractures) generated in revenue service were installed into a section of tangent track at FAST. The transverse growth of the cracks was monitored by both ultrasonic means and portmortem counting of the growth rings which developed under the FAST reversing train operation. A description of some of the results of this experiment has been given by Orringer et al (40). The growth period of transverse defects from 10-15 percent of head area to 65 percent of the head area varied from 30 MGT to 55 MGT depending upon prior revenue service conditions.

One rail sample from this experiment and five other rail samples having only revenue service exposure were examined in great detail at Battelle, Columbus Laboratories. The results are documented by Rice et al (41). Their examinations included fractography of the crack surfaces, metallography of the metallurgical microstructure near the region of crack initiation and microhardness testing of transverse sections. From their observations and knowledge of typical service induced residual stresses, the authors have proposed a hypothesis for shell and detail fracture development which will be presented in the second part of this paper.

Typically shells initiated at a depth of 0.3 to 0.4 in. beneath the surface at the bottom edge of the work hardened zone. In one specimen, there was a slight indication of a possible lens of softer material at the depth of shell initiation. But in all other specimens there was no indication of a lens of softer material. Dispersive x-ray indications were found of nonmetallic inclusions containing calcium, aluminum, and titanium in the vicinity of shell initiation sites. In some cases the indications were very strong, but in one specimen exhibiting a step on the shell surface, the dispersive x-ray indication of nonmetallic inclusion presence was less definite. Those inclusions which were associated with shell initiation were thought to be exogenous (entrapped slag). One of the specimens exhibited the presence of white etching layer on the shell surface. But in the case of other shells no mention of white etching layer was made.

CONCLUDING REMARKS

The literature shows that typically, the shell lies beneath and approximately parallel to that part of the running surface which carries most of the wheel loads. It's depth usually is between 1/4 in. and 3/8 in. (6 to 10 mm); it occurs in close proximity to the demarcation between plastically deformed and undeformed metal. It is found more frequently in the high rails of curves. In many cases (but not all), it is believed to be initiated internally. The fact that it occurs beneath the depth predicted for maximum contact shear suggests the contribution of another stress system which is believed to be the residual stress system induced by plastic deformation. The shell sometimes can exhibit a prodigious tendency to run along the rail for any feet. Frequently, the shell does *not* have an associated transverse component. The shell itself does not pose a direct threat to rail structural integrity. But the occasional, distressing tendency to turn to a transverse defect (detail fracture), which does threaten structural integrity, is adequate justification for the considerable effort which has been expended to understand the shell behavior better.

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Working from these observations a number of venturesome individuals have sought to model the initiation and growth of the shell/TD. Their efforts have progressed far enough to suggest that there are a number of practical approaches to the control of defect occurrence which can be employed. More of this will be said in Part II.

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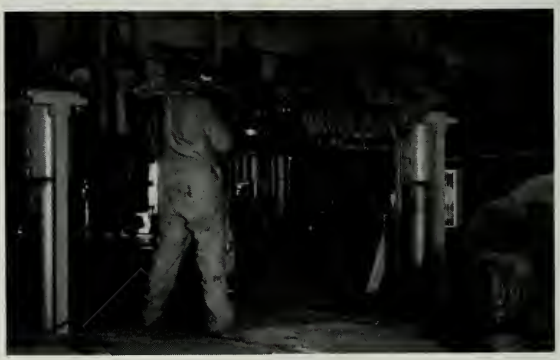

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MEMOIR

Milton Pikarsky
1924-1989

Milton Pikarsky, son of Abraham and Celia Pikarsky, was born in New York City on March 28, 1924. In 1944, upon his graduation from the City College of New York with the degree of Bachelor of Civil Engineering, he entered the U.S. Navy. Upon leaving the naval service in 1946, he began his transportation career as an assistant engineer with the New York Central Railroad. In 1956, he left the railroad to become a consulting engineer. In 1960, Pikarsky entered public service when he became Engineer of Public Works for the Department of Public Works, City of Chicago. Four years later, he was appointed Commissioner of Public Works to head that agency. In this position, Pikarsky was responsible for engineering and operating much of Chicago's infrastructure including the airports, streets, bridges, public buildings, water supply and sewerage systems. As Commissioner of Public Works, Pikarsky and his agency were also responsible for the design and construction of new rapid transit lines for the Chicago Transit Authority.

In 1973, Pikarsky left Public Works to become Chairman and C.E.O. of the Chicago Transit Authority, the second largest rail transit property in North America. From 1975 to 1978, Pikarsky was appointed the first Chairman of a new agency, the Regional Transportation Authority which oversees the C.T.A. and suburban bus and rail operations in the Chicago area. In 1978, he left the public sector to become Research Professor and Director of Transportation Research at the Illinois Institute of Technology. In 1985, Milton Pikarsky returned to the City College as Distinguished Professor of Civil Engineering and Director of the Institute of Transportation Systems, the positions he held at the time of his death on June 14, 1989.

Milton Pikarsky was elected to membership in Chi Epsilon, Tau Beta Pi, Sigma Xi, and received several other honors. He was the author of two professional books and several published articles. He was a member of the A.R.E.A. since 1952 where he served on Committee 8 — Concrete Structures and Foundations, and more recently on Committee 17 — High Speed Rail.

Milton Pikarsky is survived by his wife Christine, and his children Joel and Amy. His passing has left a void in the field of engineering and transportation and he will be missed by his many friends and colleagues.

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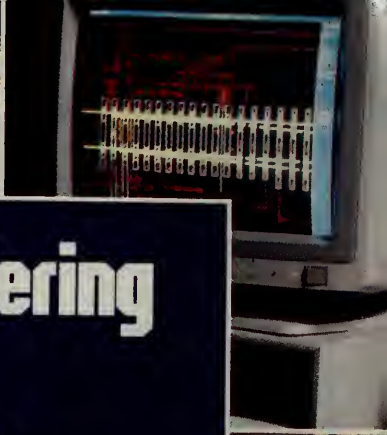

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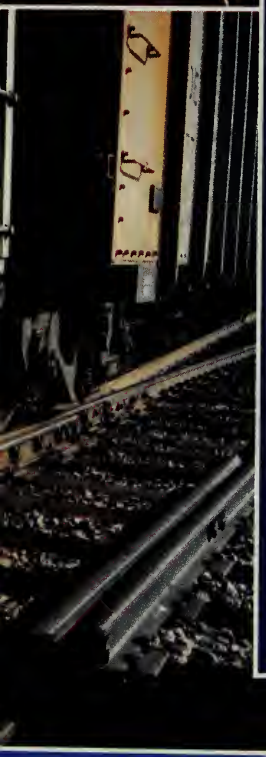

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Front Cover Photo: B.N. coal train heads south on Orin line south of Gilette, Wyoming, August, 1989.

Rear Cover Photo: Three unit coal trains await departure to south, north of single track section south of Bill, Wyoming on Orin line of B.N., August 1988.

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Photo 1 — Chicago & Northwestern unit coal train heads south near milepost 108 on B.N. track with 150 mgt/year.

150 MGT Annually on B.N. Single Track Line

Coal from Wyoming's Powder River Basin continues to be the source of traffic setting new records for annual gross tonnage over a single track railroad line. Five years ago, in the AREA October 1984 Bulletin (No. 697), there was a short article on the achievement of 120 million gross tons (mgt) annually on the single track Burlington Northern line between Alliance and Lincoln, Nebraska which at that time was believed to be the highest rate of tonnage moving over a substantial length of single track line any place in the world.

Photo 2 — Double-tracking under construction near milepost 96 on B.N. Orin line, August, 1988.

Photo 3 (above) shows rail grinder in operation on line with 150 mgt/yr and Photo 4 (below) shows rail immediately upon completion of grinding.

This record has now been exceeded on a section of the Burlington Northern's Orin Line, with runs from near Gillette, Wyoming south to near Douglas, Wyoming. About 150 mgt per year is handled between a point at milepost 72.5 known as East Logan and M.P. 117.7, Shawnee Junction. Of this 45.2 mile stretch, there are 27.7 miles of single track. This single track section handles, in addition to the traffic of the Burlington Northern, unit coal trains of the Chicago and Northwestern which operate on trackage rights over this segment.

A five mile single-track section of this line, between mileposts 93.2 and 98.2, is in the process of being double-tracked as shown in photo no. 2, where the embankment has been widened and geotextile fabric laid for the second track. Rail life has been lengthened significantly by rail profile grinding as indicated in B.N. System Chief Engineer Glavin's article in the October 1989 AREA Bulletin. Photo no. 3 shows a grinding train on the line and photo no. 4 shows the rail immediately after its passage.

Photo 5 — C & NW unit train on 150 mgt/yr section near Milepost 113.

The track structure on this line is concrete ties using the McKay fastening system. These ties were installed in 1988 and 1989, replacing the timber ties with which the line was originally built in 1979. Rail section is mostly 136 RE, with some 132 RE. The Chicago & Northwestern trains have been running over the Burlington Northern Northern tracks in this section since their new line into the area was completed to Shawnee Junction in 1984. Since one loaded 100-car unit coal train and the empty return operating each day for a year constitutes about 6.3 mgt, it can be seen that 150 mgt annually is the equivalent of 24 100-car trains in each direction per day.

With tonnage this intense, many time blocks are needed for maintenance of the track and this concentrates the traffic additionally. Natural randomness of traffic, especially when two different railroads are involved, leads to the bunching of trains, as is shown in the photo on the back cover, where three trains comprising around 330 cars of coal are awaiting passage onto a single track line segment at its northern end. It is anticipated that about 5 miles per year will be double-tracked, leading to the double-tracking of this entire single track area by about 1994.

Photo 6 — B. N. unit train heads south at milepost 107 onto single track section carrying 150 mgt.

TIE CONDITION INSPECTION A CASE STUDY OF THE FAILURE RATE, MODES AND CLUSTERING (AAR REPORT R-714)

By: D. D. Davis*
P. Chow**

1.0 INTRODUCTION

Understanding the process of tie failure is the key to effective tie maintenance planning. This report examines the failure rates, failure modes, and distribution of failures in track (spatial distribution) for mainline track. While this study is limited in scope, the concepts developed are widely applicable. This study is one component of a wide-ranging research program in track maintenance planning. The overall program is guided by the AAR Track Maintenance Research Committee.

The Track Maintenance Research Committee is composed of AAR member railroad representatives, AAR personnel, railroad research and/or regulatory organizations, AAR contractors, and members of AAR affiliated organizations. The Committee is divided into three working groups; each specializing in a major track component:

— Ballast and Subgrade
— Crosstie
— Rail

The Crosstie Working Group is responsible for research concerning crosstie maintenance and maintenance planning, including the contents of this report.

The purpose of this study is to investigate the processes of tie failure under mainline operating conditions. A series of four annual tie condition inspections were conducted on three one mile segments of mainline track. The segments are located on Conrail's Chicago-Buffalo-Albany mainline (formerly NYC) at Canastota, New York. Exhibit 1 lists the site characteristics. Mileage limits under test are: (1) MP 266-267, track 1; (2) MP 267-268, track 1; (3) MP 269-270, track 2.

The tie condition inspections provided data on the tie condition (a subjective rating of tie serviceability), tie failure mode (e.g. split, decay, broken, etc.), and tie location (i.e. failed tie clustering). The three year sequence of inspections allowed tie failure and failure mode occurrence rates to be determined.

In addition to the Working Group tie condition surveys, Conrail conducted a similar survey (failed tie count only) as part of another study in 1983. This additional information gives an extended view of tie failure rates for mile 269-270.

The railroad has conducted a different kind of condition survey of the sites in 1986. The local Conrail maintenance personnel marked ties for renewal and replaced them. The Working Group was able to compare its failure criterion with Conrail's tie removal criterion on two of the three sections. Ultimately, the tie renewal marking is the only condition survey that matters to train operations on the line. The tie renewal by Conrail allows us to compare the Working Group theories against actual practice.

The condition surveys provide a largely subjective opinion of tie condition. In order to supplement this opinion with an objective measurement of tie performance, a plate cutting survey was taken for a one mile section in 1987. These measurements provide quantitative results of tie performance in one critical area: the rail-tie interface.

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**Professor of Forestry, University of Illinois

Exhibit 1. Inspection Site Characteristics.

Rail Section	136# NYC Continuous Welded Rail. Rail Milling: Lackawanna, 1964
Tie Plates	14 inch Double Shoulder, Canted
Rail Anchors	Channel Loc 4 anchors per tie on every other tie
Track Spikes	Standard Cut Spikes 6 spikes per tie (2 gage side hold down)
Ballast Section	12″ plus under ties
Ties	7″ x 9″ Treated Oak (3,249 per mile)
Annual Tonnage	Approximately 20 MGT/track
Traffic Direction	Bi-Directional
Time Table Speeds	75/60/50 MPH
Construction Dates	MP 261.5 to MP 268.5 = 1965 MP 268.5 to MP 271.0 = 1964
Site Identification	# 1 = Track 1, MP 266.0 − 267.0 # 2 = Track 1, MP 267.0 − 268.0 # 3 = Track 2, MP 269.0 − 270.0

2.0 DATA COLLECTION

Data Collection consisted of both visual observations and objective measurements. A series of tie condition surveys were conducted over a four year period. The procedures and inspection criteria used are discussed in section 2.1. Plate cutting measurements were also performed on one of the inspection sites in 1987. The procedures used in this operation are described in section 2.2.

2.1 Tie Inspection

The visual condition surveys were conducted in order to determine the condition of each tie, the mode(s) of deterioration found in each tie, (if any), and the clustering tendency (or physical distribution) of failed ties in track.

The ties were guarded into three condition categories; good, marginal, and failed. The definition of a good tie is a tie which is satisfactorily performing its functions of holding gage and alignment without any visible signs of distress. The definition of a marginal tie is a tie which is showing signs of deterioration, yet does not meet the criterion of a failed tie. A marginal tie is a tie that could be (and most likely would be), left in track during renewal by a tie gang. The following criterion were used to determine tie failure for each distress type:

1) Decay — decayed wood to a depth of 3 inches; any loose spikes.

2) Plate Cut — plate cutting to a depth greater than the plate edge thickness.

3) Splitting — any split which extends under a tie plate and extends through the thickness of the tie. Evidence of ballast working up (or down) through the split.

4) Broken — any complete break and any partial break visible from the surface.

5) Spike Killed — any tie with loose spikes and mechanical deterioration of the spike hole. Evidence of spike killing must include horizontal transverse spike hole elongation. Indicators of spike killing include plate cutting, wide gage, worn rail and frequent rail changes.

6) Ring Shake — a separation of the wood along one or more growth rings. For this survey, ring shakes were reclassified as split or decayed. Approximately 70 percent were classified as decay and 30 percent were classified as splitting.

The inspection team consisted of two tie graders and a recorder. The procedure involved a visual inspection of each tie by the two graders. Each grader walked a rail, inspecting each tie from one end to the opposite rail. The two inspectors rated the condition of the tie, first; then, each determined the dominant deterioration mode. If the graders gave different conditions and the same mode, a second appraisal was conducted. If the two graders gave different conditions and different modes, the most severe condition and corresponding mode were used. Exhibit 2 is a copy of the data collection form used.

Also noted in the condition survey was the integrity of the spike, plate, tie fastening. Each line spike was tested for tightness by a tap from the inspector's boot. Each tie with at least one loose spike has been recorded as having loose spikes.

2.2 Plate Cutting Measurements

Plate cutting measurements were made on mile 267 during the 1987 condition survey. Every tenth tie in the segment was tagged and measured using the AAR plate cutting measuring device. This device, shown in Exhibit 3, has electronic transducers and is attached to a datalogger for automatic data storage.

The device works by measuring the difference in elevation between the top of the tie plate at a known distance from the edge and the top surface of the tie adjacent to the plate. Factors which can affect the accuracy of the measurement include: plate thickness tolerance, plate slope and warping, tie surface damage, and the presence of debris such as lubricants, oils, sand, etc.

The device is capable of measuring thicknesses as small as 0.001." However, the allowable tolerance in plate thickness as specified by AREA is 1/32".[1] Thus, the device's practical limit of precision is in the range of 0.1" to 0.01."

3.0 RESULTS

Survey results have been compiled and tabulated for tie condition, failure mode, failure rates, failure clustering, and plate cutting. Also included for comparison are the 1986 Conrail tie renewal "survey" results for the same sections.

3.1 Condition Survey Results

The tie condition results indicate that even high speed, high tonnage lines can be operated successfully with some "bad" ties in track. The Working Group's definition of a failed tie is very broad; much stricter than the Federal Railroad Administration (FRA) safety criteria. Yet, as Conrail's tie renewal shows, the crosstie component of the track structure has a fairly large (redundancy) factor of safety compared to other components. That is, the line can be operated with a relatively high number of failed or defective components. It must be kept in mind that the line surveyed is a main line, maintained to a high standard (i.e. FRA class 5). Ties called failed (i.e. marked for renewal) by the railroad meet Conrail's *maintenance* criteria for such trackage. These criteria are well above FRA safety criteria. The track surveyed was at all times maintained to standards well above FRA minimum requirements.

Exhibit 2. Inspection Data Collection Form.

CROSSTIE SURVEY DATA SHEET

DATE: _____ TRACK: _____

SITE: 1:25 M.P.1: ___ M.P.2: _____

CONDITION	MODE		SP.STR.		
0) GOOD	0) CND=GOOD	4)SPK	0)TANGENT	4) GRADE CROS	8) ILL TIE
1) MGN	1) DECAY	5)BROKEN	1)CUR(>2)	5) TURNOUT	9) OTHER
2) FAILED	2) PLT CUT	6)RING SHK	2)TRK ON BR	6) ABNRM TIE-SZ	
	3) SPLIT		3)BR OV TK	7) ABNRM TIE-SPC	

COND	MODE	SP.STR	COND	MODE	SP.STR
0.1.2	0.1.2.3.4.5.6	0.	0.1.2	0.1.2.3.4.5.6	0.

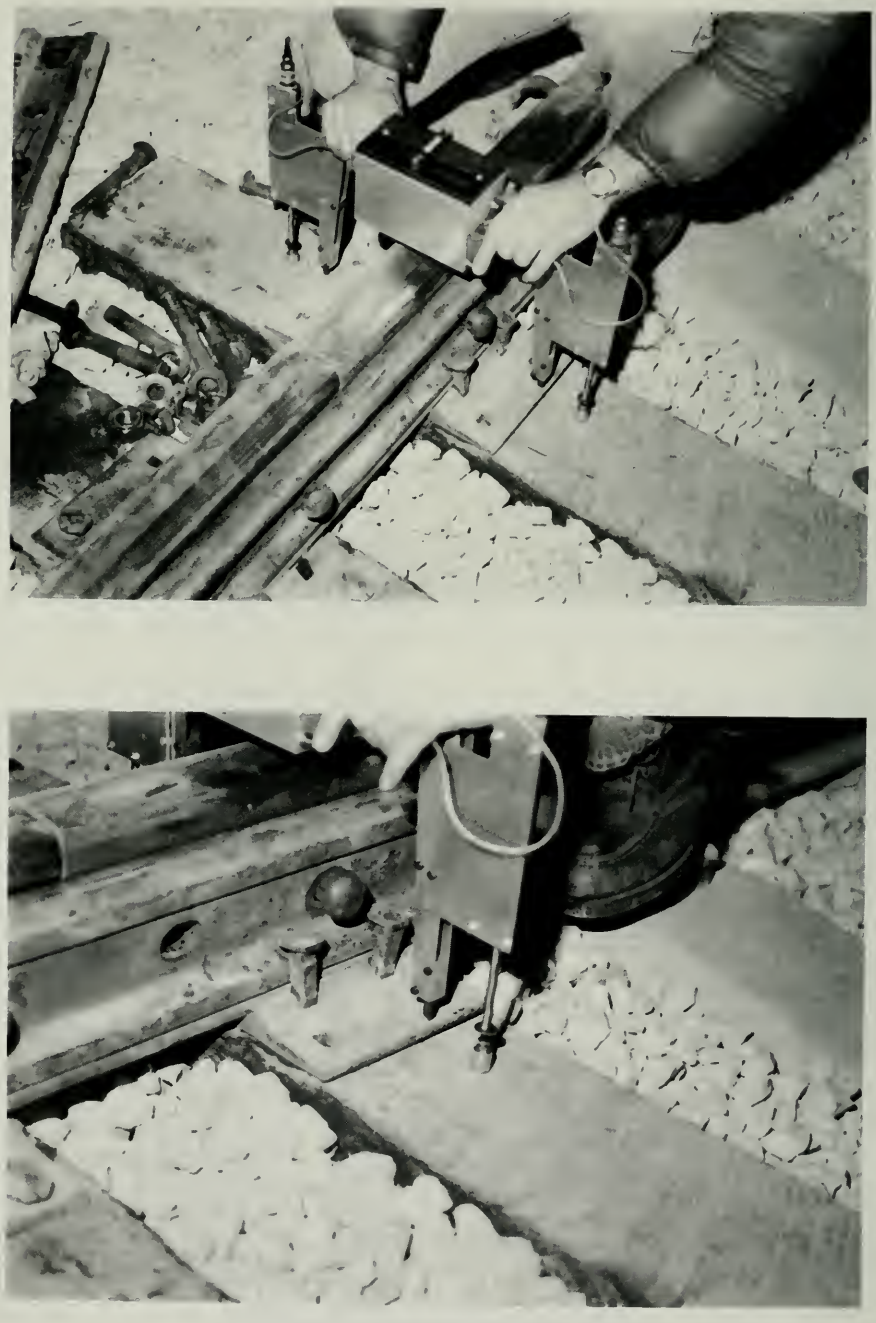


Exhibit 3. Plate Cutting Measuring Device.

The large numbers of failed and marginal ties present in each mile are due to the age of the ties in track. The average service life of oak ties in this region is 25 to 35 years. The ages of the ties surveyed were 19 and 20 years in 1984 and 22 and 23 years in 1987. According to the USDA Forest Products Curve[2] the ties are reaching the ages when the failure rate becomes highest. Exhibit 4 lists the annual condition survey data, while Exhibits 5, and 6 show the failure data plotted for each year in track. Exhibit 5 gives the actual condition of the ties at each survey. Exhibit 6 gives the cumulative failures in track at each survey. The differences between the two are the 1987 values which reflect the tie replacements made after the 1986 survey.

Exhibit 4. Condition Survey Results

Canastota Tie Survey Data

Site 1 - M.P. 266-267 **Year Installed: 1965**

Survey	# Ties	% Good	% Marginal	% Fail	Sum	Lambda
1983					0.0	
1984	2803*	40.0	51.0	9.0	100.0	0.12
1985	2898*	43.7	28.6	27.7	100.0	0.37
1986	2644*	39.9	25.4	34.7	100.0	0.39
1987+	3122	63.9	23.0	13.1	100.0	0.18

Site 2 - M. P. 267-268 **Year Installed: 1965**

Survey	# Ties	% Good	% Marginal	% Fail	Sum	Lambda
1983						
1984	3179	49.3	41.3	9.4	100.0	0.15
1985	3299	50.3	23.5	26.2	100.0	0.33
1986	3274	52.1	21.0	27.0	100.0	0.33
1987+	3188	55.0	25.1	19.9	100.0	0.22

Site 3 - M. P. 269-270 **Year Installed: 1964**

Survey	# Ties	% Good	% Marginal	% Fail	SUM	LAMBDA
1983				5.9	5.9	
1984	3176	35.5	46.0	18.5	100.0	0.29
1985	3324	27.5	38.8	33.7	100.0	0.44
1986	3280	51.6	17.1	31.3	100.0	0.45
1987+	3314	56.8	24.9	18.3	100.0	0.27

*Approximately 300 poorly performing ties eliminated from survey results (See Appendix 6.2)
+ 1987 survey results reflect the 1986 tie installation

The homogeneity of the sections pose a tie maintenance problem to the railroad. If the ties follow a Forest Products Curve service life distribution, the railroad will need to replace the majority of the ties within a 10-15 year period. The railroad is just now beginning the heavy renewal period where large tie installations will buy little track condition improvement compared to "typical" sections (i.e. ties of a mix of different ages). Conventional methods will be inefficient in selecting, marking, and replacing ties over this section.

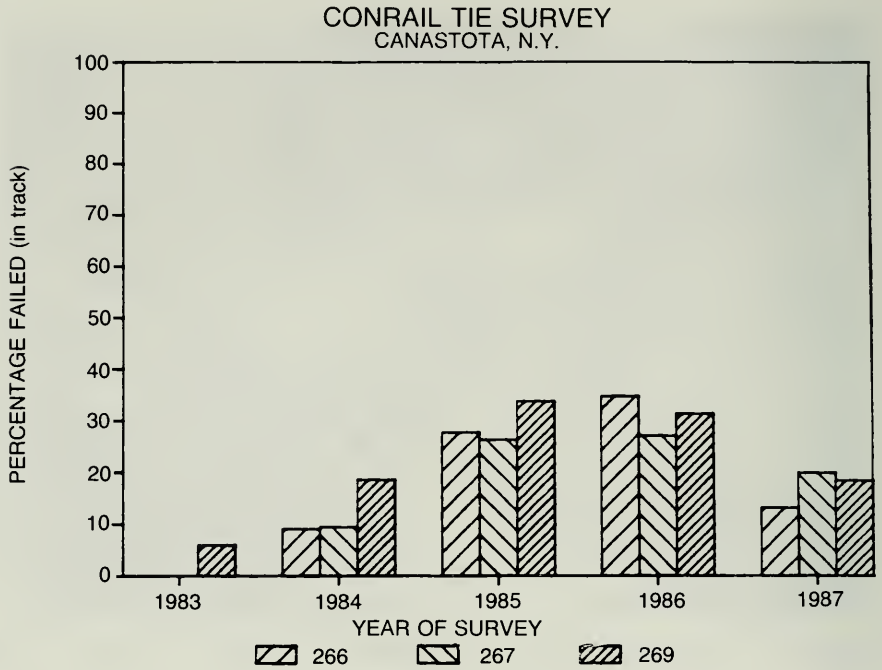


Exhibit 5. Annual Tie Condition.

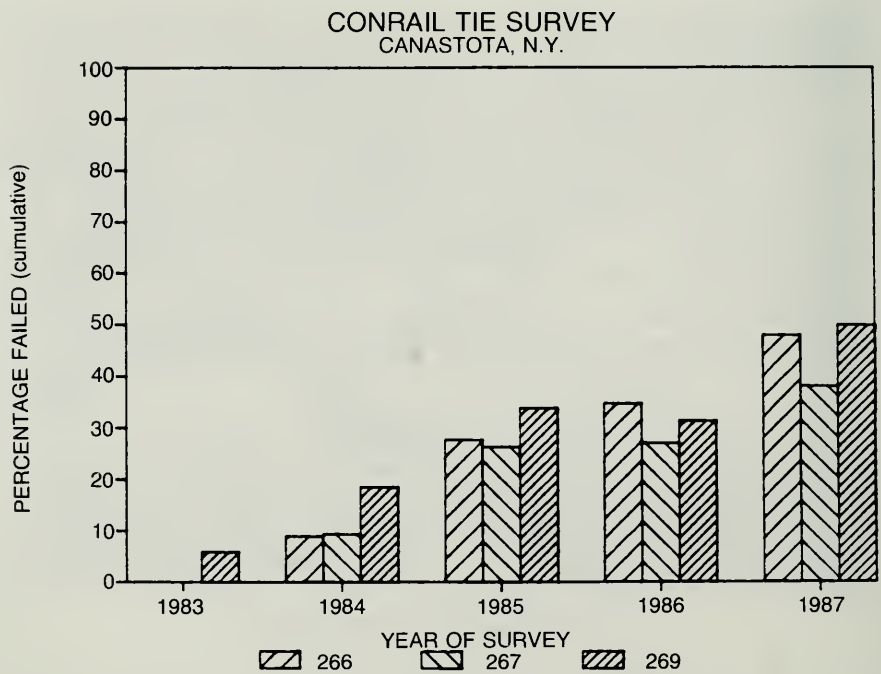


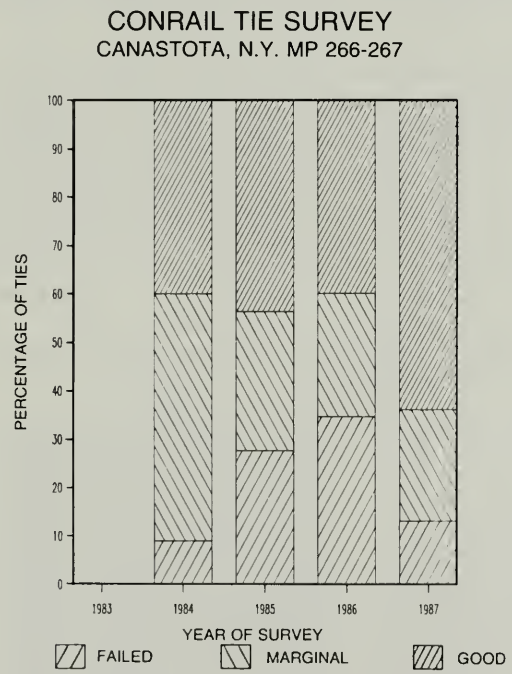
Exhibit 6. Cumulative Failure vs Time.

The Conrail tie renewals averaged about 800/mile or about 25 percent of the ties in track. This threshold is in the typical range of values seen for heavy mainline trackage. It appears to be a compromise between the maximum tolerable bad tie density for train operations or surfacing and the minimum bad tie density for efficient tie installation/service disruption.

Over the four years of tie inspections there were some changes in the perspectives of the tie inspectors. It appears that the 1985 inspection may have been too critical when compared to the 1986 inspection; or the 1986 inspection was too lenient.

Comparison of inspection records for the two years show shifts in failure criteria and marginal tie criteria. Most affected were the marginal tie criteria (Exhibit 7-9). The number of marginal ties has decreased as the inspectors have become more experienced. The definition of a good tie was also broadened to include more (formerly) marginal ties.

Exhibit 7. Tie Condition Survey Results: Site 1—Mile 266.



The whole body of data collected does show the general trends of failure modes and rates. The time series of inspections allows one to direct aberrations such as the variance in inspection criteria. Other items noted during the inspections include:

— Weather conditions affect the inspectors judgement/criteria. Wet weather tends to close up checks, but also emphasizes soft spots in ties. Extremes of temperature or rain will shorten the

Exhibit 8. Tie Condition Survey Results: Site 2—Mile 267.

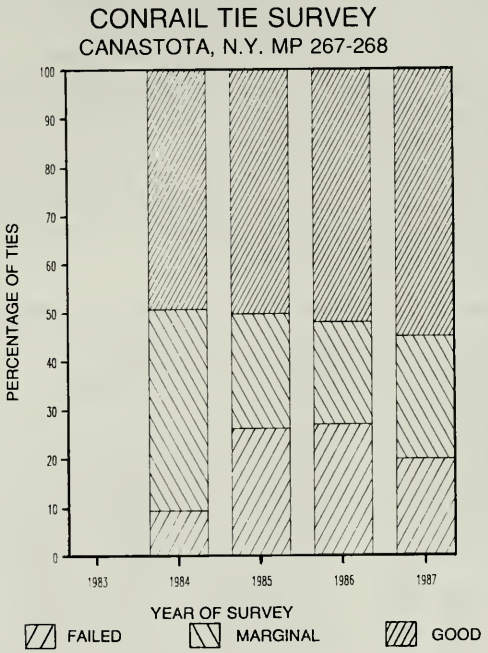
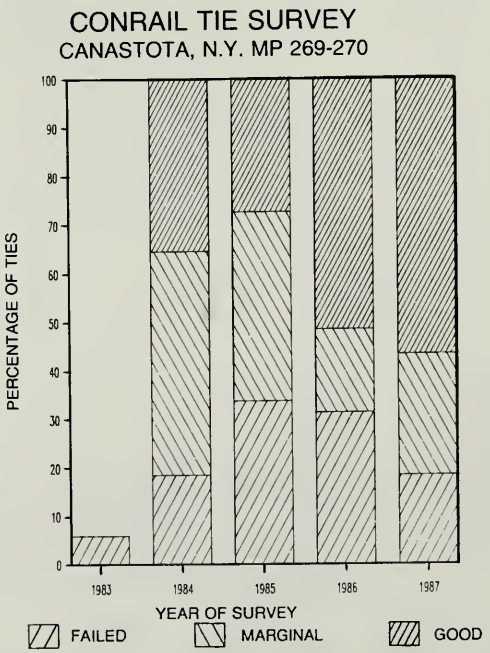


Exhibit 9. Tie Condition Survey Results: Site 3—Mile 269.



inspectors dwell time and attention span. This will result in a much smaller number of marginal ties.

— Tie inspectors will vary between themselves and amongst themselves. Just as no two tie inspectors will mark a mile the same, a tie inspector's criteria may change over four years. Strict adherence to a predetermined criteria, using the same inspectors, is required to have continuity.

— The Tie Working Group's inspection criteria is too conservative for most practical railroad requirements. The Working Group criteria differed most in the marking of plate area defects.

— Often, the selection of failure mode had to be arbitrary. Many ties exhibit more than one type of deterioration. It is frequently unclear to an inspector viewing a failed tie which failure mode occurred first or "caused" the failure. For example, ties will suffer extensive decay after failure by splitting or plate cutting opens the tie, exposing untreated wood. Decay is not the cause of failure but over time it becomes the dominant mode of failure. With the big mechanized gang method of tie renewal, ties which have reached the Working Group's definition of failed may remain in track for years until a sufficient number of failures have accumulated to justify a renewal.

The bad tie survey data can be used to predict future bad tie counts or, more importantly, to predict future maintenance requirements. This can be done simplistically with a linear extrapolation; however, a better approach is available. With a sufficient tie installation history, one can more effectively predict future renewal requirements. The United States Department of Agriculture's Forest Products Curve gives one the capability to more accurately predict tie requirements.

The Forest Products Curve (FPC) is a graphical representation of the distribution of tie failures by tie age as developed from tie replacement records.[2] The curve is normalized to average tie life. Since we are still uncertain about the actual average tie life of the test ties, plots of percent bad ties vs. tie age are shown in Exhibits 10, 11 and 12. FPC's for several average life values are shown. Conrail

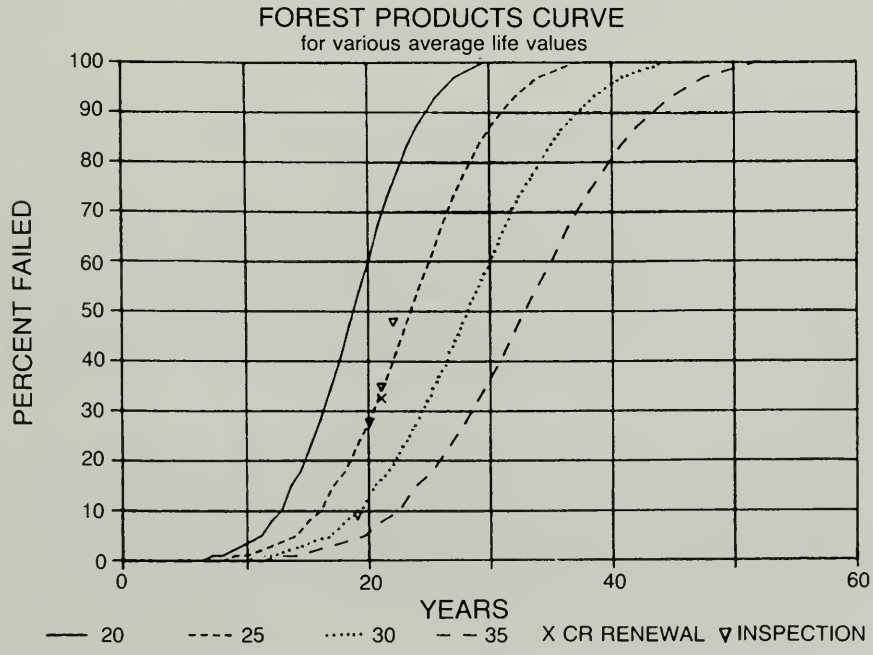


Exhibit 10. Cumulative Failures vs Years in Track: Mile 266.

personnel commented that 35 years would be a typical tie life for ties of that environment and vintage. If one uses these values as the average tie life then one can see that the failure rate projected for the next five years at each mile is quite high. The ties are reaching the age at which failures are quite common.

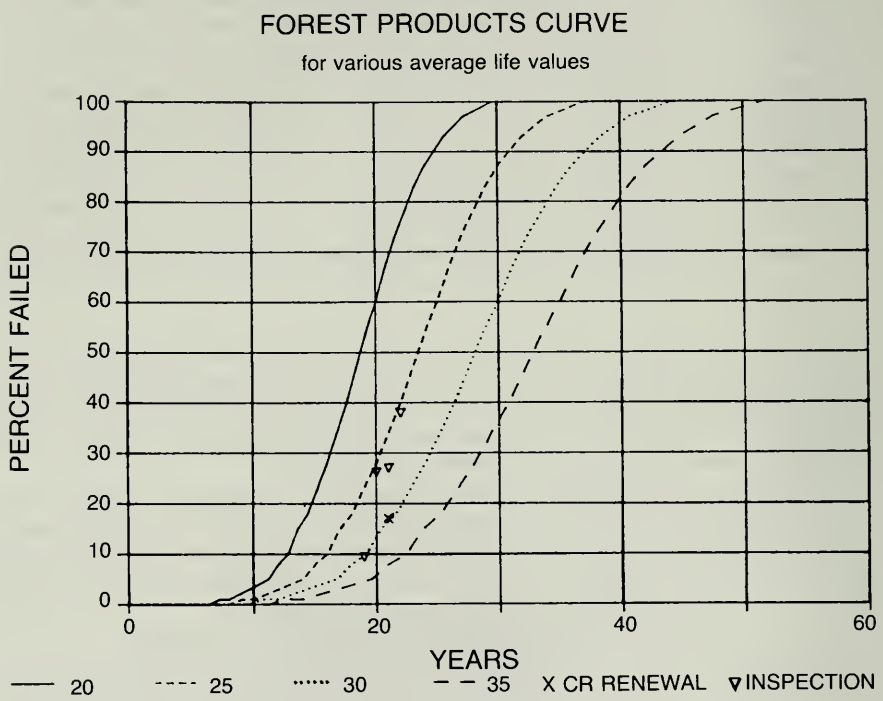


Exhibit 11. Cumulative Failures vs Years in Track: Mile 267.

The Conrail experts' estimates of 35 year service lives for these ties appears to be reasonable and correct. While it is too soon to be certain of the ties average life, the data falls into the range of the 25 to 30 year Forest Products Curves. The test site ties are failing in a more narrow range of ages than the Forest Products Curve distribution. This can be seen by the steeper slope of the actual data as it "crosses over" FPC's of shorter average life values. This is probably due to the more uniform conditions of the test sites. The unusual circumstance of all ties having the same treatment, age, and load histories may account for the more uniform failure distribution.

3.2 Failure Mode Survey Results

The failure mode survey results proved to be very enlightening in the sense of how little we know about tie failure. The process of tie failure is a very complex one; with climate, traffic, support, maintenance, and other component condition factors all interacting. The choice of failure mode was, sometimes out of necessity, arbitrary. Often a tie would exhibit several modes of failure. The survey procedure was to assign a single (dominant) failure mode to each tie.

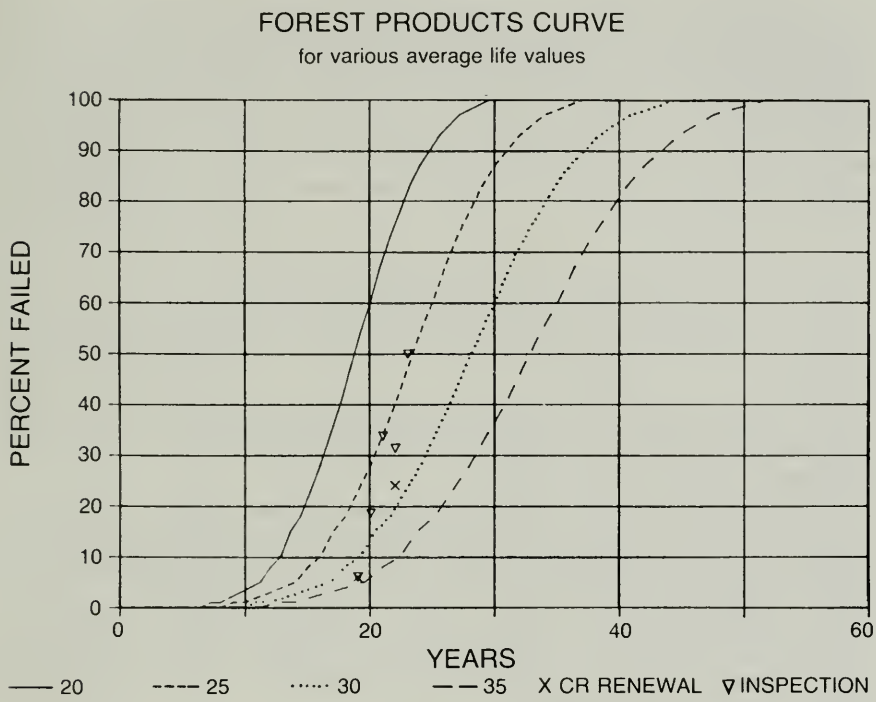


Exhibit 12. Cumulative Failures vs Years in Track: Mile 269.

In the case of loose spikes a second piece of information is available. Ties that had loose spike(s) were detected by testing the spikes for looseness. These ties were marked in the survey log as having loose spike(s). The spike test was generally preceded by the visual condition evaluation. Thus, the predominant failure mode from the visual survey and the presence of loose spikes were both noted in the log.

Rarely did a tie that was graded as good have loose spikes. However, many marginal ties were downgraded to failed when the presence of loose spikes was noted.

The Tie Working Group, in reviewing the survey results, concluded that virtually all of the ties having loose spikes did not fit the classical or survey definition of a spike-killed tie. The spike holes had little or no elongation. Lateral forces are not considered to be the cause of spike loosening at the site.

The occurrence of loose spikes appears to be environment related. Many of the loose spikes are "down," sitting on the rail base. Spike material or tie material or both have been lost to allow the spike to loose holding power to the extent that the head sits on the plate. This indicates a chemical/weathering or decay (biological) deterioration process.

Other spikes are "up," sitting well above others. Some of these are loose or "wobbly." These spikes were probably pulled up by the action of the rail and tie under traffic. The rail "uplift" force pulls up on the spike in the precession wave as the rail rises before the passage of a wheel or between the first and second trucks of a vehicle.

Loose spikes were almost always associated with another failure mode. Typically, evidence of decay/weathering and/or plate cutting were seen with loose spikes. There is no clear causal relationship present between decay, weathering and loose spikes. But it is likely that they are mutually beneficial to each other (and most assuredly are detrimental to tie performance) in acting together to promote tie deterioration.

The results of these surveys (Exhibit 13) show that tie plate/spike area deterioration is pervasive at this site. However, it does not appear to be the major "cause" of failure (if one defines cause of failure as the reason for removal.)

Most of the ties with loose spikes appear to be chemically deteriorated or weathered. The spikes are loose but there is little evidence of spike hole elongation. The spike loosening could be due to the vertical forces imparted by rail moving up and down as wheels traverse across the tie site. Most of the ties are still "working" (i.e. moving up and down with the rail) since rarely were all line spikes loose on one tie. Due to the random nature of the location of the loose spikes, mechanical action is not the most likely cause.

Some of the loose spikes were associated with decay failures. A large proportion of the ties with loose spikes, typically 50-70 percent, were listed with decayed/weathered as the reason for removal. The rest were classified as split or plate cut.

There were very few ties which fit the "classical" description of a spike-killed tie. A spike killed tie is one having one or more spikes loose, spike holes elongated, and extensive loss of gage and/or gage restraint capacity. While there is some evidence of a *slight* dynamic gage widening throughout the site, there is no evidence of reduced gage restraint capacity amongst the loose spike ties.

One would not expect to find many spike-killed ties on the type of line studied in Canastota. The line is tangent with minimal grades. Track surface is generally very good . . . The line supports high speed passenger and freight service. And, the rail is the original rail at the site. There have been no rail relays or transpositions on this segment.

Exhibit 13. Tie Plate/Spike Area Deterioration.

Mile	Number of Ties Surveyed	Number of Ties Failed	Number of Ties w/Plate Area Damage	Percent of Total Failed
266	2644	910	291	32
267	3274	884	430	49
269	3280	1024	450	44

Thus, most of the conditions which cause spike-killing are not present. The repeated high lateral loading present in curved track is not present in this site. There are occasional high lateral loads of a more random (in terms of location) nature due to truck hunting. The high speed trains on this line frequently have empty, and in the cause of intermodal equipment, loaded cars "hunting."

The dominant modes of failure at these sites are Decay/Weathering and Splitting. Other modes of failure present are Plate Cut, Broken, Spike Killed and Ring Shake. Decay/Weathering is the most prevalent mode of failure with from 50 to 85 percent of the total failures. Exhibits 14-18 show the tie failure mode distributions from each yearly survey.

The time dependency of tie failure mode occurrence has been speculated by the Tie Working Group. It has been suggested that the Forest Products Curve can be partitioned by defect types. Exhibit 19 shows a hypothetical defect differentiated Forest Products Curve for a moderate tonnage mainline. The limited time range of our survey data precludes confirmation of this hypothesis. However, some evidence exists to support it. The incidences of plate cut ties and loose spikes was generally rising over time. One would expect this to occur over time as the wear and fatigue related defects require a large number of load cycles.

The occurrence of splits and decay/weathering are quite variable over time. One would expect splits to occur earlier in average tie life than most decay/weathering failures. The data does not show any discernible pattern in this regard. There is a trend toward increasing splits and decreasing decay/weathering. This trend is not significant because both modes of failure are usually present in the ties. The trend is caused by a shift in preference or criteria away from decay/weathering as the dominant failure mode by the inspector. This is understandable in that many of these ties probably do not have much biological decay. The combined sum (in percent) of decay/weathered and split failures has remained relatively constant.

Exhibit 14. Tie Failure Mode Survey Results.

Canastota NY Tie Site

Site 1—M.P. 266-267

		# Failed					
Survey	**Total**	**# Decay**	**# PC**	**# Split**	**# SK**	**# Broken**	**# RS**
1984	257	182	0	72	0	3	0
1985	797	566	1	230	0	0	16
1986	910	443	39	427	0	1	26
1987	400	249	10	138	0	3	8

Site 2—M.P. 267-268

		# Failed					
Survey	**Total**	**# Decay**	**# PC**	**# Split**	**# SK**	**# Broken**	**# RS**
1984	299	234	0	63	0	2	0
1985	862	696	0	163	0	3	10
1986	884	664	7	213	0	3	18
1987	663	377	16	264	0	6	11

Site 3—M.P. 269-270

		# Failed					
Survey	**Total**	**# Decay**	**# PC**	**# Split**	**# SK**	**# Broken**	**# RS**
1984	611	391	0	214	0	6	0
1985	1121	873	12	232	0	4	13
1986	1024	855	13	156	0	0	0
1987	595	435	18	149	0	2	4

3.3 Failed Clustering Survey

The condition surveys conducted for each tie provided a sufficient sample for investigating the physical distribution of failures in track. By studying the spatial distribution of failures in track, one may determine the extent and character of the effect if any, of a failed tie on the adjacent ties. If a failed tie does have a detrimental effect on its neighbors, failed tie "clusters" (i.e. groups of two or more consecutive ties) would occur more frequently. If there is no effect, tie failures would occur randomly (in location) with clusters also occurring randomly.

This, it is possible to check for clustering and to quantify the relative strength of the clustering tendency. AAR[3,4] has developed two parameters for measuring clustering tendency:

1) P—the probability that a tie has failed (i.e., the percentage of bad ties).
2) Lambda—the probability that a tie has failed, given that an adjacent tie has failed. If the ties fail in a totally random manner Lambda and P should be equal.

Exhibit 15. Tie Failure Mode Distributions.

Canastota NY Tie Site

Site 1—M.P. 266-267 **Failure Modes**

Survey	# Decay	# PC	# Split	# SK	# Broken	# RS	SUM %
1984	70.8	0.0	28.0	0.0	1.2	0.0	100.0
1985	71.0	0.1	28.9	0.0	0.0	1.2	100.0
1986	48.7	4.3	46.9	0.0	0.1	1.2	100.0
1987	62.3	2.5	34.5	0.0	0.8	1.2	100.0

Site 2—M.P. 267-268 **Failure Modes**

Survey	# Decay	# PC	# Split	# SK	# Broken	# RS	SUM %
1984	78.3	0.0	21.1	0.0	0.7	0.0	100.0
1985	80.7	0.0	18.9	0.0	0.3	1.2	100.0
1986	75.1	0.8	24.1	0.0	0.0	1.2	100.0
1987	56.9	2.4	39.8	0.0	0.9	1.2	100.0

Site 3—M.P. 269-270 **Failure Modes**

Survey	# Decay	# PC	# Split	# SK	# Broken	# RS	SUM %
1984	64.0	0.0	35.0	0.0	1.0	0.0	100.0
1985	77.9	1.1	20.7	0.0	0.4	1.2	100.0
1986	83.5	1.3	15.2	0.0	0.0	1.2	100.0
1987	73.1	3.0	23.5	0.0	0.3	1.2	100.0

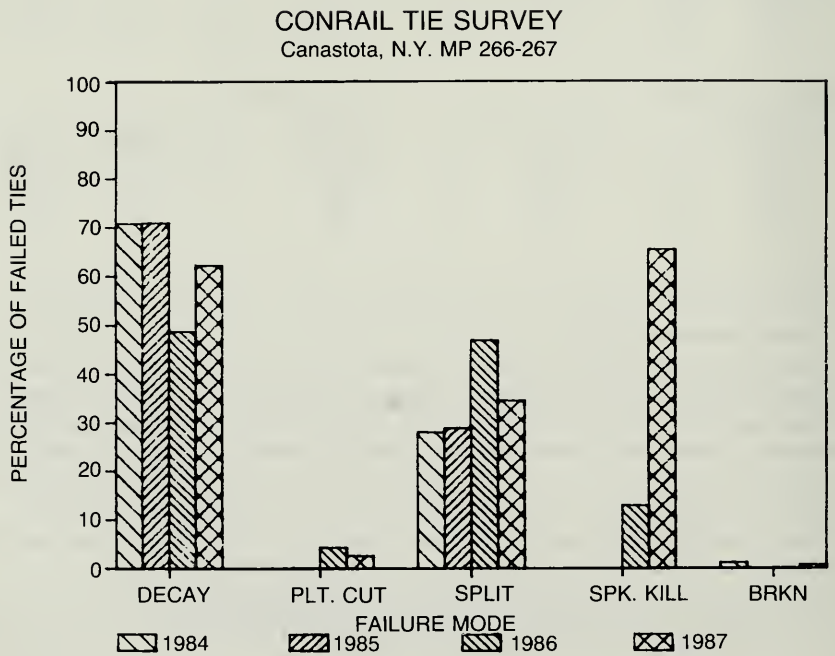


Exhibit 16. Tie Failure Mode Distribution: Mile 266.

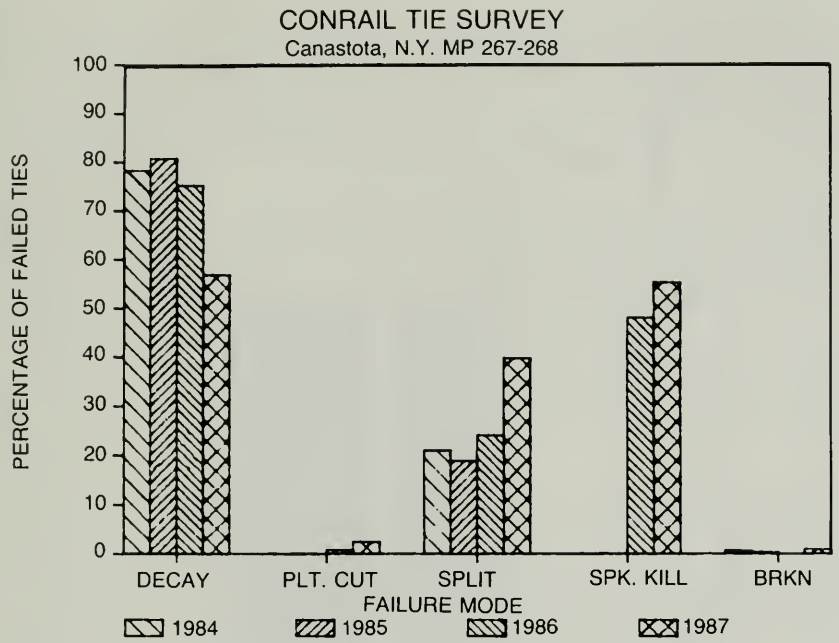


Exhibit 17. Tie Failure Mode Distribution: Mile 267.

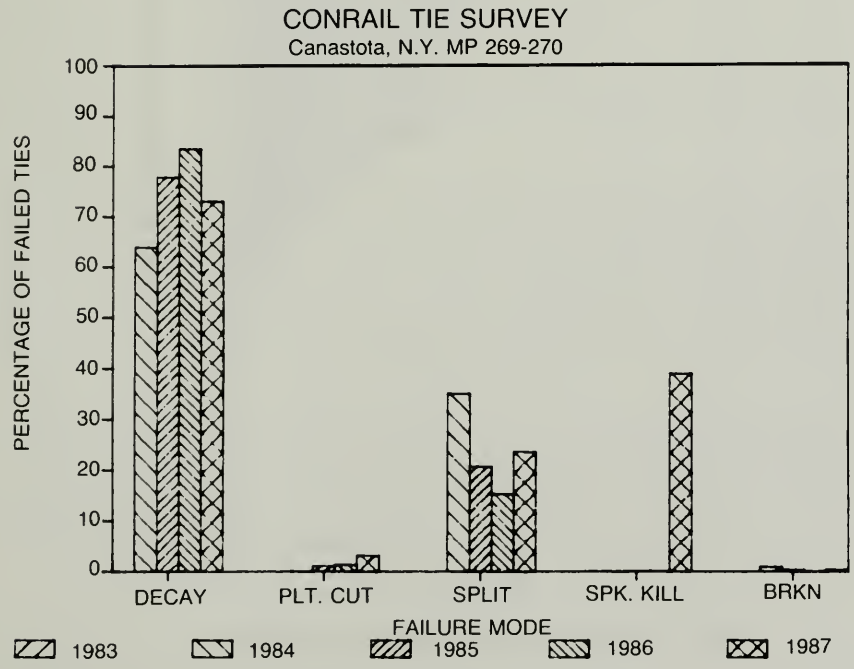


Exhibit 18. Tie Failure Mode Distribution: Mile 269.

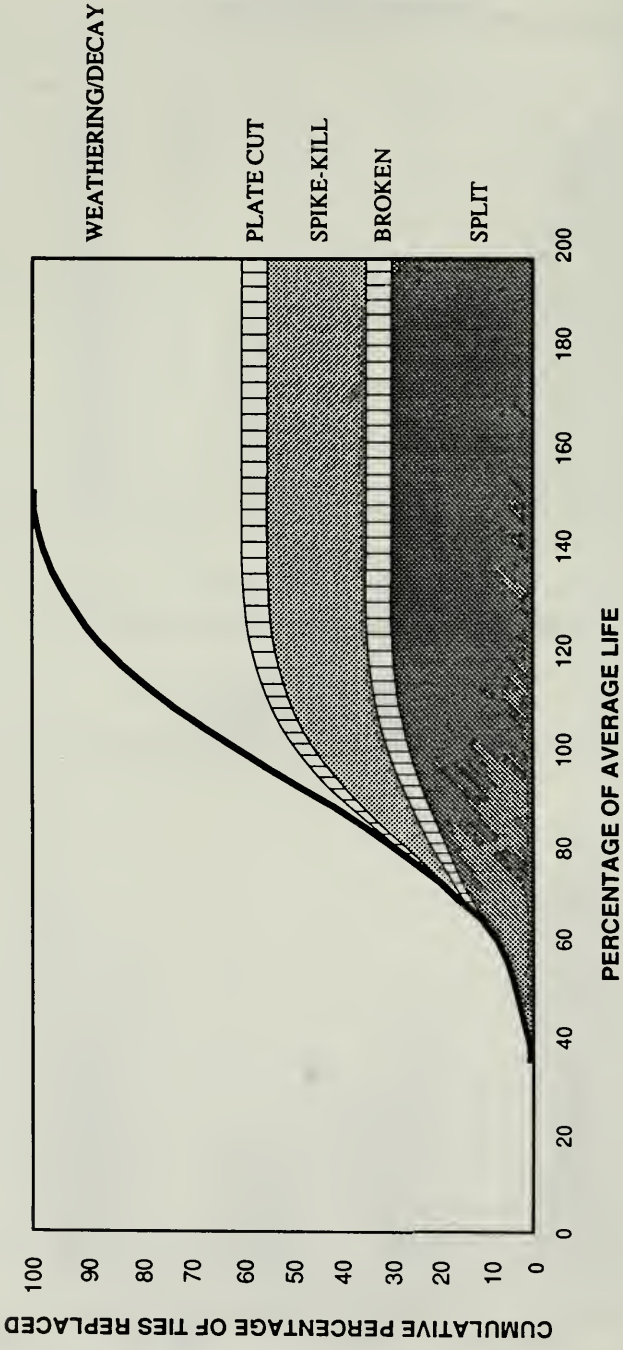


Exhibit 19. Hypothetical Defect Mode Differentiated Forest Products Curve.

Exhibit 20 lists the actual failed tie cluster distributions for each mile and each year of the survey. Also included is the distribution of clusters for various numbers of bad ties developed by a random number generator. The calculated lambda and lambda/P values are also provided. Lambda/P is a relative measure of the clustering tendency of the failed ties. Lambda/P values greater than one indicate a clustering tendency. Values less than one indicate an anti-clustering tendency.

Exhibit 21 illustrates the clustering tendency found with the test ties. Survey results for all three miles for all four years indicate a clustering tendency. This tendency generally decreases with increasing failures. This trend has been noted at other locations[4] in the past. It suggests that at least some of the ties are influenced by adjacent failures. At some point, the effect is diminished to the extent that a more random failure distribution develops. This could be the point at which environment-related failure modes account for most of the failures rather than mechanical-related failure modes.

One of the reasons for selecting this site was the known tie maintenance history. The lack of major tie renewals on the site allows us to observe tie failure distributions in track as they would naturally occur. Most sites have the confounding effects of tie maintenance policy/procedures which tend to vary with location and time.

The Canastota sites show the natural tendency for tie failures to cluster. This tendency appears to be common for mainline track and big gang renewal policies. The climate, as it affects tie failure modes is also likely to be a factor in clustering.

The largest factor, however is the maintenance policy used by the railroad. The policy of spotting-in ties (i.e. frequent replacements of small quantities of ties) tends to reduce clustering because it is aimed at breaking up clusters. Whereas, the "big gang" approach of few occurrences of replacements of large quantities of ties (e.g. 800 per mile) tends to allow clustering to occur[5].

The renewal record for the site shows that replacement ties marked by Conrail and, thus, the previously failed ties, were clustering. The Lambda/P values at renewal were: 1.13, 0.90, and 1.25 for miles 266, 267, and 269.

The post renewal survey (1987) indicates that the tendency to cluster still exists. The Lambda/P values for 1987 were: 1.40, 1.11, and 1.47 for miles 266, 267, and 269. It should be emphasized that the Working Group survey criterion of failure is not the same as the Conrail tie marker's criterion of failure.

To illustrate the effect that clustering has on tie failure distributions, Exhibits 22-24 show the actual failed tie cluster distribution and a computer generated random failure distribution for the same bad tie count. As one can see the number of clusters of three or more ties is increased when compared to the random case. These clusters are the weak points in track that cause increased vehicle and lading damage as well as an increased safety concern. Recent tests of lateral track strength[6] have shown that a single bad tie or missing tie does not materially affect lateral track strength. It is the clusters which are the weak points.

Exhibit 23 shows the results of a study of the effects of Lambda on cluster distributions. This study used the AAR Tie Renewal model to calculate cluster distributions.

For two cases, one with few failed ties and the other with many, a parametric study of the effect of Lambda on the cluster distribution was conducted. In both cases, varying the Lambda value (i.e. maintenance policy) from one half to twice the failed tie rate produces large differences in the number of clusters of three or more failed ties predicted. This range of Lambda values covers the limits seen in a large nationwide sample of track. It is the clusters of three or more failed ties which pose the greater risk of derailment and track, equipment or lading damage.

Exhibit 20. Clustering Survey Results, Actual Results and Hypothetical Random Failures.

SURVEY	MILE	NUMBER OF CLUSTERS										% BAD	LAMBDA	
SITE		1	2	3	4	5	6	7	8	9	10+	P	L	L/P
CR 1984	266	187	27	2	1	0	0	0	0	0	0	0.09	0.14	1.51
CR 1985	266	319	114	47	13	8	1	1	1	0	0	0.28	0.37	1.34
CR 1986	266	349	127	52	10	11	5	2	1	0	1	0.35	0.39	1.12
CR 1987	266	272	53	8	2	0	0	0	0	0	0	0.13	0.18	1.39
CR RENEW	266	408	159	53	18	14	5	2	1	1	0	0.35	0.39	1.12
CR 1984	267	211	34	4	2	0	0	0	0	0	0	0.09	0.16	1.71
CR 1985	267	392	120	42	21	2	2	0	0	0	0	0.26	0.33	1.26
CR 1986	267	402	131	39	13	5	2	2	0	0	0	0.27	0.33	1.21
CR 1987	267	392	94	18	3	1	0	0	0	0	0	0.20	0.22	1.07
CR 1987	267	397	94	18	2	3	0	0	0	0	0	0.21	0.22	1.07
CR RENEW	267	391	61	10	0	0	0	0	0	0	0	0.17	0.15	0.87
CR 1984	269	279	76	34	4	1	2	0	2	0	0	0.18	0.32	1.72
CR 1985	269	376	142	44	19	12	9	8	1	3	2	0.33	0.44	1.32
CR 1986	269	338	133	52	13	15	4	6	2	1	3	0.30	0.43	1.41
CR 1987	269	344	62	28	7	5	0	0	1	0	0	0.18	0.27	1.46
CR RENEW	269	395	116	25	12	0	1	1	0	0	1	0.24	0.29	1.25
RANDOM	5%	168	8	2	0	0	0	0	0	0	0	0.06	0.06	1.08
RANDOM	10%	272	29	4	0	0	0	0	0	0	0	0.11	0.11	1.03
RANDOM	15%	370	60	12	1	0	0	0	0	0	0	0.16	0.16	1.00
RANDOM	20%	435	99	16	2	1	0	0	0	0	0	0.21	0.20	0.95
RANDOM	25%	471	128	22	11	3	0	0	0	0	0	0.26	0.25	0.97
RANDOM	30%	480	159	36	15	6	1	0	0	0	0	0.31	0.30	0.98
RANDOM	35%	473	181	63	18	6	3	2	0	0	0	0.36	0.36	0.99
RANDOM	40%	471	198	77	35	8	3	2	0	0	0	0.40	0.39	0.97

Exhibit 21. Lambda/P vs P.

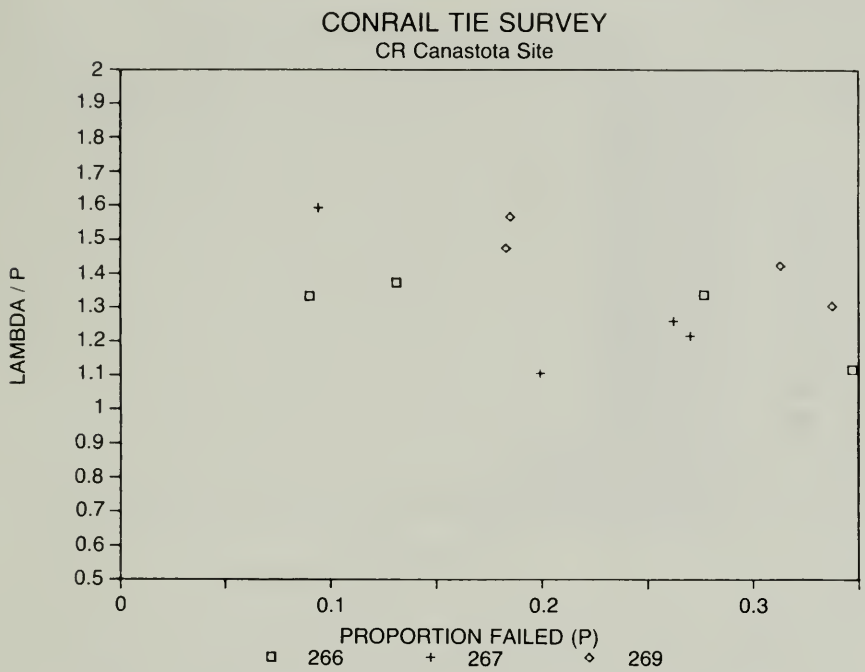


Exhibit 22. Actual and Equivalent Random Cluster Distributions: Mile 266.

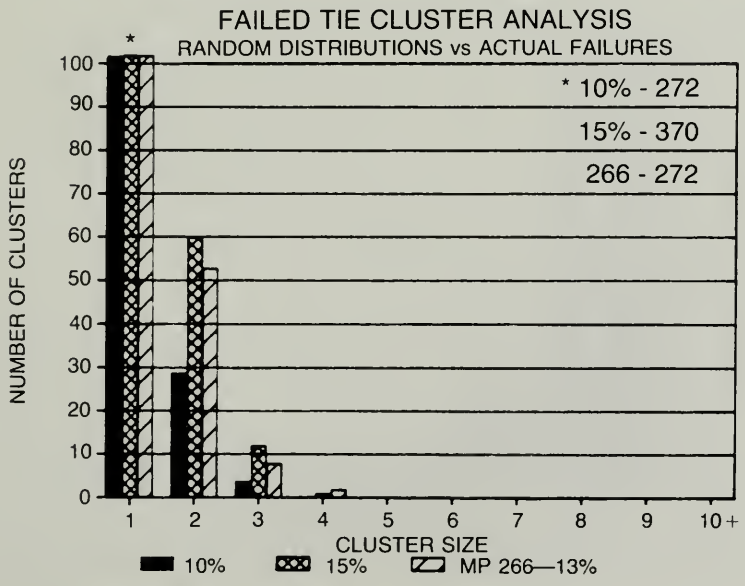


Exhibit 23. Actual and Equivalent Random Cluster Distributions: Mile 267.

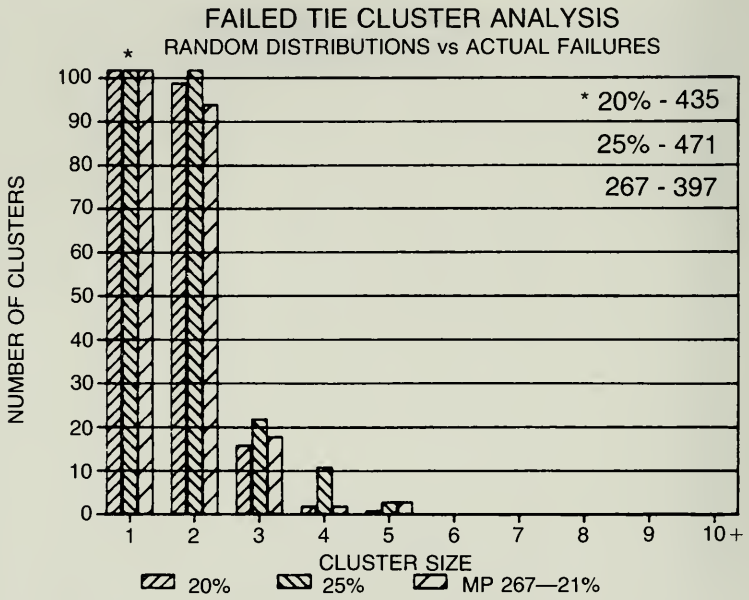


Exhibit 24. Actual and Equivalent Random Cluster Distributions: Mile 269.

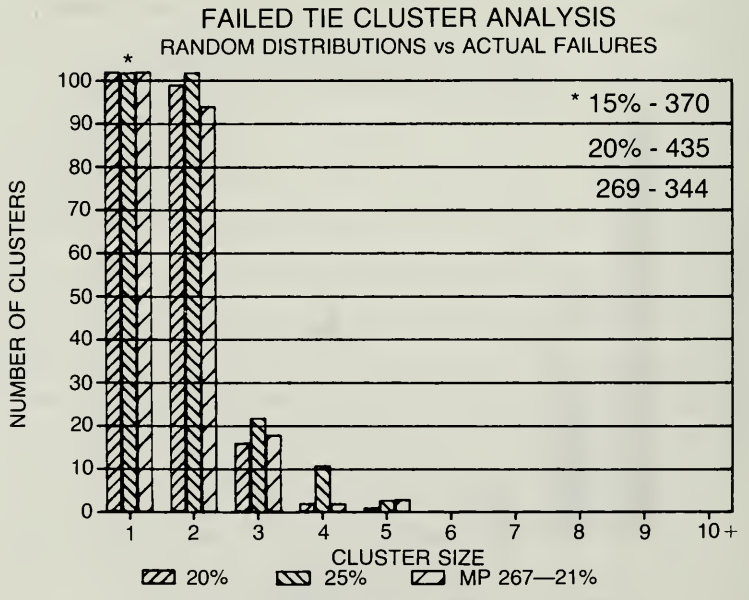


Exhibit 25. The Effect of Lambda on Cluster Distributions.

3250 ties/mile

Fraction Failed (P)	LAMBDA	L/P	Cluster Size 1	2	3	4	5	6	7	8	9	10+
.10	0.05	0.5	306	15	1	0	0	0	0	0	0	0
	0.10	1.0	275	27	3	0	0	0	0	0	0	0
	0.15	1.5	245	37	6	1	0	0	0	0	0	0
	0.20	2.0	217	43	9	2	0	0	0	0	0	0
	0.25	2.5	191	48	12	3	1	0	0	0	0	0
	0.30	3.0	166	50	15	4	1	0	0	0	0	0
	0.35	3.5	143	50	18	6	2	1	0	0	0	0
	0.40	4.0	122	49	20	8	3	1	0	0	0	0
	0.50	5.0	85	42	21	11	5	3	1	1	0	0
	0.60	6.0	54	33	20	12	7	4	3	2	1	1
	0.70	7.0	31	21	15	10	7	5	4	3	2	4
.32	0.05	0.16	949	47	2	0	0	0	0	0	0	0
	0.10	0.31	851	85	9	1	0	0	0	0	0	0
	0.15	0.47	759	114	17	3	0	0	0	0	0	0
	0.20	0.63	673	135	27	5	1	0	0	0	0	0
	0.25	0.78	591	148	37	9	2	1	0	0	0	0
	0.30	0.94	515	154	46	14	4	1	0	0	0	0
	0.35	1.09	444	155	54	19	7	2	1	0	0	0
	0.40	1.25	378	151	61	24	10	4	2	1	0	0
	0.50	1.56	263	131	66	33	16	8	4	2	1	1
	0.60	1.88	168	101	61	36	22	13	8	5	3	3
	0.70	2.19	95	66	46	32	23	16	11	8	5	14

3.4 Plate Cutting Measurements

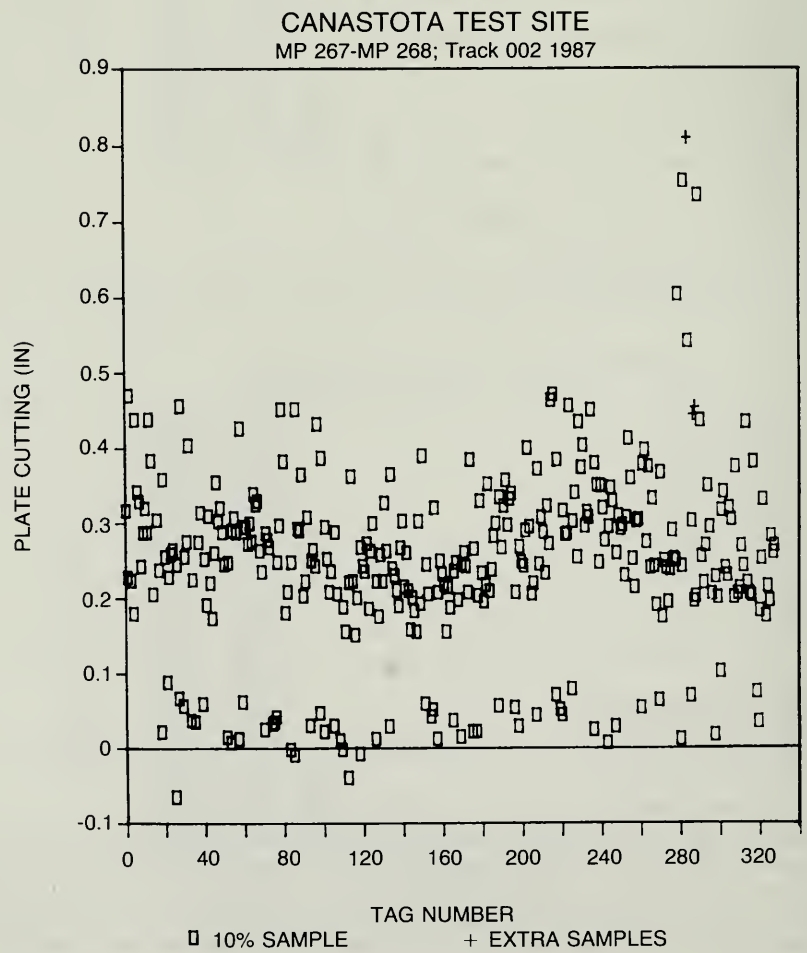
The results of the plate cutting survey are described below. The survey measured every tenth tie in mile 267 (survey site 2).

The data collected from the survey has been plotted in Exhibit 26. The tag numbers start from 001 at M.P. 267. Tag number 100, for example, is attached to the 1000th tie west of M.P. 267. The average plate cutting value plotted is the average of four measurements: a gage and field side measurement of each plate. The four measurements provided additional information on plate tilting or differential plate cutting. While these phenomenon occur frequently on curves, they did not occur much on the well maintained tangent track at Canastota. Thus, only the average value is plotted for each tie measured.

Exhibit 27 lists the relevant statistical data for the site. The average plate cutting value is 0.25". As one can see from Exhibit 26 there are two distinct groups of data. The "lower" group with plate cutting of less than 0.1" are mostly the ties which were installed in 1986. The (relatively) large initial plate cutting experienced by these ties is most likely due to "seating" of the plate in the tie. The "upper" group consists of the original ties installed in 1965. The data falls in a band of scatter from 0.2" to 0.5". Over 22 years, the average plate cutting rate was 0.009"/yr. to 0.022"/yr.

The performance of the ties, in plate cutting, has been good. Plate cutting of 0.25" over 22 years on a high speed, heavy tonnage line is not a maintenance problem. Factors contributing to this good performance are: good ballast and support, well maintained track surface, and minimal track disturbance (i.e. no rail relays and no tie renewals up to 1986). Most ties, including the hard to treat species, will have treatment penetration beyond 0.5". Plate cutting will not reach untreated wood for many years.

Exhibit 26. Plate Cutting vs Tie Number



Three additional measurements were made in an area of higher plate cutting. An area of 50-80 ties had significantly higher plate cutting (0.5-0.8") than the rest of the section. The ties with higher plate cutting were associated with a series of field welds and engine burn defects. The track surface and tie condition were noticeably poorer in this section than elsewhere. The additional amount of plate cutting is assumed to be due to the higher dynamic loading and rail movement caused by the rail surface anomalies. Ballast condition is also poor in this area. There is some evidence of pumping in this area, although it is not a major problem at this time. Drainage along the track is generally very good. Pumping is not a problem at other locations.

It is not clear what are the causal relationships between the occurrence of rail surface defects, ballast fouling/pumping, and high(er) plate cutting of ties. One theory is that the plate cutting and ballast pumping are caused by dynamic loads generated by rail alignment/geometry problems. Rail surface defects (e.g. engine burns) may also be caused by these conditions; however, they maybe caused by variations in "local" rail surface and material properties or train operations.

Another explanation of the excessive plate cutting in this area is the variability of the wood properties and/or treatment received by these ties. There is no documented treatment history for these ties. The ties were treated in an era in which we no longer have a record of tie treating procedures. Thus, the treatment could account for significant variability.

Exhibit 27. Plate Cutting Results.

Plate Cutting	Left Field	Left Gage	Right Gage	Right Field	Left Plate	Right Plate	Tie
All Ties							
AVE	0.21	0.27	0.26	0.24	0.24	0.25	0.25
STD	0.14	0.14	0.15	0.15	0.13	0.14	0.13
MIN	-0.10	-0.30	-0.04	-0.08	-0.30	-0.08	-0.04
MAX	0.93	0.88	1.15	0.99	0.91	1.06	0.87
New Ties (1986)							
AVE	-0.00	0.07	0.06	0.01	0.03	0.04	0.04
STD	0.06	0.06	0.05	0.04	0.04	0.04	0.03
MIN	-0.10	-0.30	-0.01	-0.08	-0.30	-0.08	-0.30
MAX	0.28	0.14	0.30	0.15	0.28	0.30	0.30
Old Ties (1965)							
AVE	0.26	0.31	0.30	0.29	0.28	0.29	0.29
STD	0.10	0.12	0.13	0.12	0.10	0.11	0.09
MIN	0.05	-0.22	-0.04	0.06	-0.22	-0.04	-0.22
MAX	0.93	0.88	1.15	0.99	0.93	1.15	1.15

3.5 Failed Tie Strength Tests

During the 1986 tie renewal of the test section approximately 24 specimens were obtained for testing. These ties were selected by AAR personnel from the ties marked for replacement by Conrail in track 001 between MP 266 and MP 268. The ties were shipped by Conrail to Urbana, Illinois for testing at the University of Illinois, Department of Forestry Laboratories.

The ties chosen were selected based on the Tie Working Group's visual inspection of 1986. Six ties each were selected from the good, marginal, and failed categories. An additional six ties were selected from the group of 300-400 ties near MP 266 that were performing badly. These ties were excluded from the Working Groups condition surveys, as discussed in Appendix 6.2.

Each of the ties in these four groups were tested for several relevant strength properties. This data was collected for the following purposes:

1) To help develop strength-age-environment relationships for wooden crossties in service in North America.
2) To determine the relationship between visual inspection condition and objective measures of tie strength/condition.

Strength tests were conducted by the University of Illinois following procedures developed for analysis of artificially aged specimens.[7] Following is a brief description of each test performed:

Whole tie tests:

1) Static bending Test—a three point bending test was conducted on each specimen. A 60-inch span was used to simulate a "center-bound" tie.

Tie piece tests—each tie was sawn into 18 inch long specimens for testing:

1) Compression perpendicular to grain load (24,000 lbs.) was applied to specimens using a 115 lb. rail on a 13″ x 7-3/4″ steel tie plate.

2) Surface hardness and hardness modulus—a two inch diameter steel ball was imbedded 0.25 inches into the crosstie specimen.

3) Pre-bored (1/2″) spike joint: a) the spike insertion load, b) the lateral load resistance with a displacement of 0.2 inches, and c) the direct spike withdrawal load were recorded.

The results of the bending tests are given in Exhibits 28 and 29. The maximum static bending load results in Exhibit 28 illustrate some interesting points. The twenty year old ties have suffered a significant strength loss as compared to a new tie. The unusual tie group, which all could agree were failed, had only 50 percent of the new tie strength. Also, these ties had lower strength than the other 20 year old ties. The ties classified as good, marginal and failed (but marked as failed by Conrail) had very similar strength values (within 10 percent of each other).

The bending modulus of elasticity values show a similar pattern to the bending load values; with the marginal ties performing better than the good or failed ties (Exhibit 29).

The surface hardness values are shown as retention values (percentage of new tie value) in Exhibit 30. The ties have retained a large percentage of their original strength on the exposed middle surface of the ties. However, in the plate areas the hardness has decreased to 55 percent (of original) for good ties and marginal ties, to 40 percent for bad ties and to 25 percent for the unusual ties.

Exhibit 28. Maximum Static Bending Load.

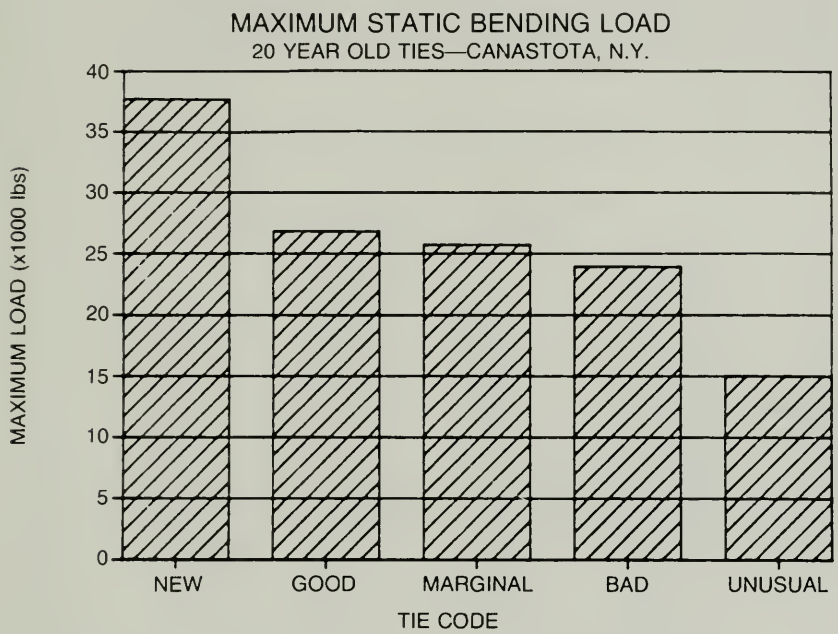
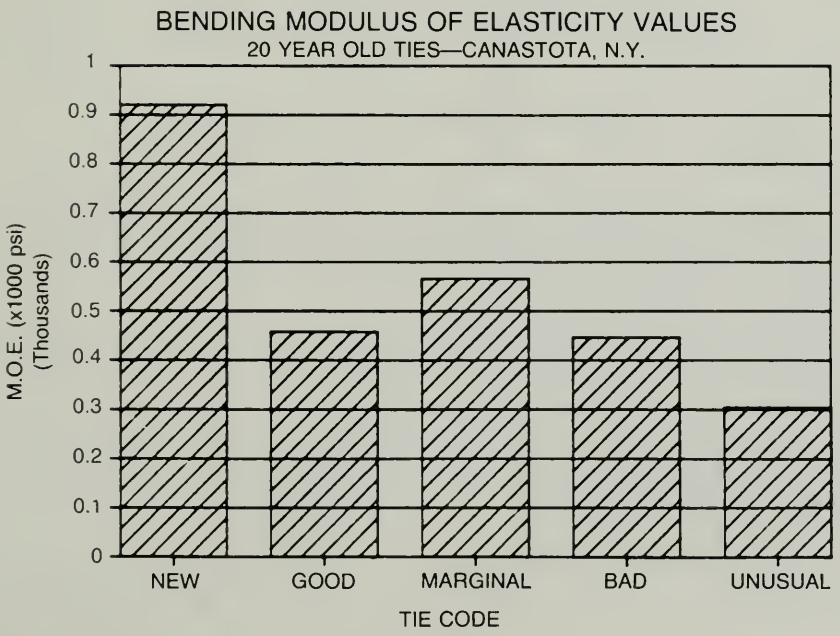
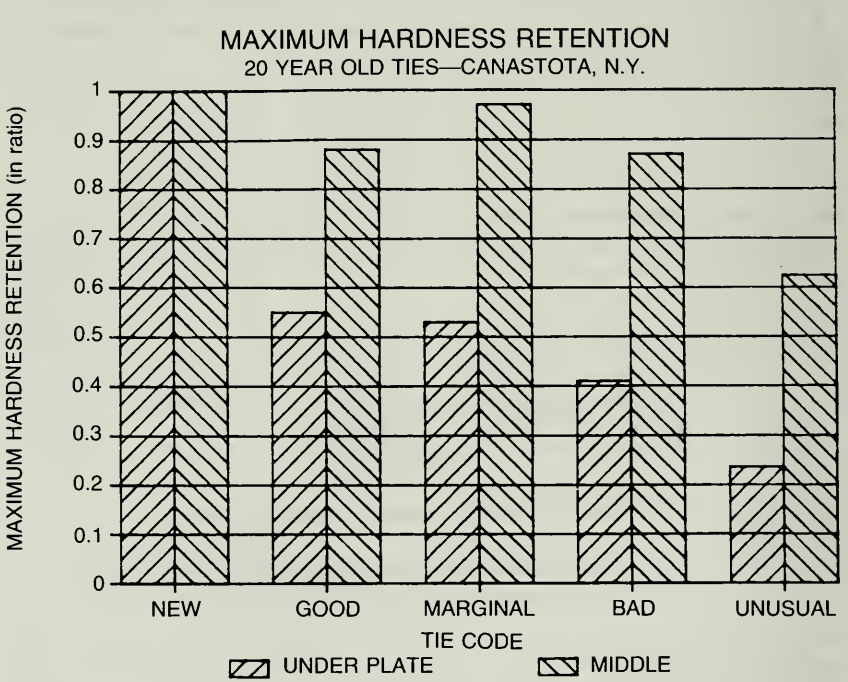


Exhibit 29. Bending Modulus of Elasticity Values.



The compression modulus results are shown in Exhibit 31. As with the hardness results there is a difference between middle-of-tie and plate area results. The middle-of-tie retention values are higher than the plate area values, but not as high as the corresponding hardness values. The plate area values are very similar to the previously reported hardness values: good and marginal: 50-60 percent, bad: 40 percent, and unusual: 27 percent.

Exhibit 30. Maximum Hardness Retention.



The spike test results show spike withdrawl resistance losses of 33 to 80 percent for the spectrum of tie conditions. However, the lateral strength losses were more consistent, with a range of 46 to 65 percent. This result suggests that the failures were not caused by the lateral rail movement typically seen in curved track spike killing.

There is a clear distinction between the spike resistance values of the unusual, bad, and good ties; but, the distinction between good and marginal ties is not clear. Thus, the marginal ties may truly be good ties that show signs of incipient failure. Both vertical and lateral spike stiffness properties are essential to rail rollover resistance and gage restraint.

Exhibit 32 is a summary of the results of the mechanical tests performed on the ties removed from the test sites. The results are presented as strength retention values in comparison to test values of ten new oak crossties supplied by Conrail.

Exhibit 31. Compressive Modulus of Elasticity Retention.

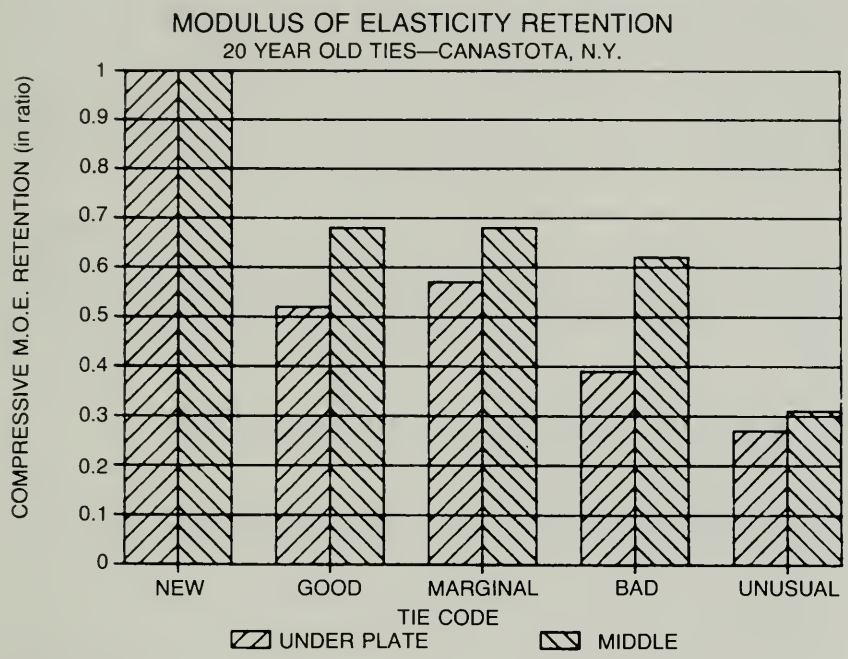


Exhibit 32. Tie Strength Retention Levels.

STRENGTH RETENTION OF 20-YEAR OLD OAK TIE (CONRAIL)
BASED ON RESULTED OBTAINED FROM 10 NEW CROSSTIES (PERCENT).

CODE[1]	STATIC BENDING		COMPRESSION (MOE)	FACE HARDNESS	SPIKE		LATERAL SPIKE RESISTANCE
	MOR	MOE			IN	OUT	
New Oak	100	100	100	100	100	100	100
0 (5)[2]	71[3]	71	61 (P)[4] 80 (NP)[5]	60 (P) 100 (NP)	81	67	54
1 (6)	68	74	67 (P) 80 (NP)	57 (P) 95 (NP)	81	62	50
2 (6)	63	63	46 (P) 70 (NP)	44 (P) 93 (NP)	64	41	40
3(5)	40	41	32 (P)	29 (P)	47	15	35

[1] Canastota, NY: 0 - Good
1 - Marginal
2 - Bad
3 - Unusual

[2] Number of ties tested.

[3] Value in percent.

[4] P-Under the tie plate area.

[5] NP-Not under tie plate area.

The results show that ties in track suffer from a loss in strength associated with aging or weathering. The 20 year old ties had losses of 30-50 percent of the nominal new value for various strength properties. Also, tie surface condition and mechanical test results may not correlate. The differences in strength between good and bad ties tested were relatively small (and often not statistically significant).

The differences between plated and non-plated areas is substantial. The results indicate that deterioration is greatly increased in plated areas. The causes cannot be determined from the test results. Iron degradation, biological decay, and mechanical failure (i.e. crushing) are all more likely to occur in the plated area.

The Working Group visual condition survey does appear to be able to correlate visual condition with some strength properties. If one considers marginal ties to be good then the process can distinguish a good tie from a bad tie consistently. The bad ties differ from the unusual ties in the extent of their failure and perhaps also the mode(s) of the failure. The unusual ties would still be replaced if they were in a branchline; whereas, some of the bad ties would not. They are the "discretionary maintenance" ties.

4.0 CONCLUSIONS

The following conclusions can be drawn from the Canastota tie analysis:

1) Ties installed out-of-face deteriorate more uniformly than in normal track, but failures increase rapidly when nominal tie life is approached.

2) In the absence of tie renewals, the locations of occurrences of tie failures are not random but tend to cluster.

3) The tendency for failures to cluster in track decreases with time (i.e. increasing failure count) or tie age.

4) The assignment of the mode of failure to a crosstie through visual inspection is highly subjective. Tie failure is a complicated process that typically involves the interaction of several failure modes. Several possible modes of failure may be present.

5) Natural or environmental deterioration was apparently the predominant factor influencing failure of ties in this 20 MGT, high speed tangent site.

6) Strength loss of ties in track due to aging or weathering can be quite significant. Losses of 30 to 80 percent of the nominal new tie strength were found after 22 years in track. The most significant losses occurred in the tie plate area.

7) Plate cutting is not a significant problem on this tangent high speed track. After 22-23 years in track, these oak ties averaged only 0.29 inches of plate cutting.

8) At this site, loose spikes were due to environmental causes such as decay, weathering or chemical attack. There was little evidence of loosening of spikes through mechanical action.

5.0 REFERENCES

1. American Railway Engineering Association, Manual for Railway Engineering, Volume 1, Chapter 5, p. 5-1-3.

2. Maclean, J. D., "Percentage Renewals and Average Life of Railway Ties," Forest Products Laboratory Report No. 886, Madison, Wisconsin, 1965.

3. Hargrove, M. B. and Shade, K., "Development of a Statistical Model for Tie Clustering," Unpublished Report, Association of American Railroads, 1982.

4. Davis, D. D., Tie Planning: A Progress Report of the Track Maintenance Research Committee, Working Paper No. WP-127, Association of American Railroads, 1987.

5. Minutes of the Tie Working Group (9/07/87), AAR Track Maintenance Research Committee.

6. McCown, R., "Initial Results of the FRA Split-Axle Device Field Tests," presented to the Tie Working Group, (5/14/87).

7. Lewis, S. L., et al., Durability of Wood Crossties (Phase 1), R-702, Association of American Railroads, 1987.

8. Burns, D. R. and Roney, M. D., Tie Inspection to Achieve Economies in Tie Renewal, Working Paper No. WP-135, Association of American Railroads, 1988.

6.0 APPENDIX

6.1 A Comparison of Tie Inspectors

As previously noted, there were some differences in the Tie Working Group's 1986 tie inspection and the Conrail tie inspector's 1986 marking for renewal. The reasons for the differences can be traced to the different purposes and methods used for the inspections.

The Conrail tie inspector's purpose is to mark ties for replacement on a line that has already been chosen for tie renewal. He may or may not have a quota of ties to fill per mile or segment. He may or may not have a time consideration (i.e. when is the next renewal cycle). He has but two categories for ties: good and bad.

The Working Group, on the other hand, has the luxury of a third category of condition, marginal, and the knowledge that their decisions are not binding. The Working Group's purposes are to grade ties and determine causes of failure. There are no quotas for the Working Group.

By trying to determine failure causes, the Working Group spends more time on each tie. The Working Group surveying one to two miles per day, going over each tie thoroughly with two inspectors. The Conrail tie renewal marker undoubtedly inspected at a faster rate. The industry average is 6 miles/day.[8]

There is no doubth that the Conrail tie renewal inspector is more efficient and experienced than the Working Group in tie inspection. But, the Working Group's examination of each tie is more rigorous and complete. The Working Group's methods are more likely to find internal or below-surface defects.

Exhibit 33 lists the tie marking and condition survey results for each mile in 1986. The results indicate agreement within 20 percent, for the failed tie count, for two of the three miles. The third mile (267) shows a 58 percent difference between the two inspections.

In all cases, the Tie Working Group (T.W.G.) survey had higher failed tie counts. This is expected as the survey criteria are known to be overly strict.

Exhibit 33. Comparison of 1986 Inspections.

MP	T.W.G. Surveyed	Conrail Marked	Difference	Percentage Difference
266-267	1024	890	134	15.06
267-268	884	560	324	57.86
269-270	910	781	129	16.52

A direct comparison tie by tie was made for the two miles (266 and 267) on track 001. This was possible due to the tie renewal marking preceding the T.W.G. inspection. The T.W.G. inspection of mile 269 on track 002 was conducted earlier in the year; prior to tie marking. Of the ties marked for renewal, approximately 79 percent were also graded as failed by the T.W.G. Only 1-2 percent were graded as good. About 20 percent were graded as marginal; a luxury the tie marker did not have. Exhibit 34 lists these results.

A further look at the marginal ties that were marked for renewal shows a high proportion of split ties. Exhibit 35 lists the failure mode distribution of these marginal ties. A higher proportion of split ties might be expected in the marginal ties because of the decay after failure phenomenon discussed in Section 3.1. However, the split percentages present in this group of ties suggest a difference in grading criteria for splits amongst the two inspection methods.

Exhibit 34. Distribution of Ties Marked for Renewal by Conrail 1986 Survey.

MP	Marked	Good	Marginal	Failed	% Good	% Marginal	% Failed
266-267	890	17	177	696	1.91	19.89	78.20
267-268	560	6	110	444	1.07	10.64	79.29
1987							
269-270	781						

It appears that the T.W.G. is more lenient on splits.

Exhibit 35. Analysis of Marginals by Failure Mode.

MP	Marked	Split	Decay	Other	% Split	% Decay	% Other
266-267	177	156	21	0	88.14	11.86	0.00
267-268	110	86	17	7	78.18	15.45	6.36

The T.W.G. and renewal marking surveys also differ on the matter of loose spike ties. The T.W.G. survey tested each line spike on each tie; whereas, the Conrail inspector, it is assumed relied on a visual survey only. The discrepancies in bad tie counts correlate with the number of loose spike ties found in the T.W.G. survey. (see Exhibit 13).

6.2 Elimination of Poorly Performing Ties from the Mile 266 Survey.

Visual examination of the ties in Mile 266 revealed a section of unusual looking ties. These approximately 300 ties were performing quite poorly in comparison to the rest of the ties. A large percentage of these ties, 60 percent in 1984 and 78 percent in 1985, were graded as failed by the Working Group. These ties exhibit considerably more plate cutting, decay and loose spikes than the rest of the ties in the section. The ties appeared to be more grey colored than the others, with a loss of material between rays of still existing material.

The visual appearance differences in these ties were corroborated by the condition survey results. Exhibit 36 shows the 1984 survey results with and without the poorly performing ties. The ties in the poorly performing section had 60 percent grade as "failed" by the Working Group's rather strict criterion. The rest of the section had only 9 percent graded failed by the same criterion.

Also the cluster distribution of the failed ties in this section is markedly different from any other section encountered by the Working Group. There were large clusters of "failed" ties within this short section. Virtually all ties within a cluster had the same failure modes and appearance.

This unusual condition suggests that the failures were due to some material, seasoning, or treatment effect that was unique to this group of ties. In light of the large differences in performance between this group of ties and the rest of the ties in the three mile survey, the ties were removed from this and all future surveys.

Exhibit 36. 1984 Condition Survey Results—Mile 266, including the Poorly Performing Tie Section.

	Mile 266 Entire Section	Mile 266 Poorly Treated Section	Mile 266 W/O Poorly Treated Section
Number of Ties	3079	267	2803
Number "Failed"	418	167	251
Percent "Failed"	13.6	62.5	9.0
Number of Clusters of Size			
1	213	26	187
2	31	4	27
3	8	6	2
4	4	3	1
5	3	3	0
6	1	1	0
7	2	2	0
8	2	2	0
9	3	3	0
10	0	0	0
11	0	0	0
12	1	1	0
13	1	1	0

It appears that Conrail agreed with the Working Group's assessment of the condition of these ties. The 1986 tie gang replaced most of these ties. Thus, the new replacement ties were included in the 1987 condition survey.

COMPILATION OF FIELD MEASUREMENTS ON BALLAST STATE OF COMPACTNESS

By: Ernest T. Selig*

Parameter Definitions

The state of compactness of ballast in track may be represented by dry density, DD (ratio of particle dry weight to total ballast volume), void ratio, e (ratio of void volume to particle volume) or porosity, n (ratio of void volume to total volume, in percent). These three parameters are interrelated by

$$n = \frac{e}{1 + e}$$

and

$$DD = \frac{G}{1 + e} \quad WD$$

where G = particle specific gravity, and WD = density of water.

Purpose

Porosity is the best measure of the void storage space in ballast and it also is an indication of permeability. Porosity is a function of particle size and gradation as well as the processes compacting the ballast (Ref. 10). Dry density is one of the factors influencing the constraining ability of ballast, such as in the shoulders and cribs. Particle specific gravity is an important parameter in these applications as well as void space.

Method of Measurement

Density and porosity of ballast are very difficult to measure in the field. Thus very little such data exist. However, special apparatus has been devised (Ref. 11) and used to study ballast in a variety of track conditions.

Before the special apparatus was developed for measuring density, existing methods used in railroad and highway applications were evaluated (Ref. 12). These methods included rubber balloon, sand cone, oil or water replacement, and nuclear radiation. The first three of these methods were found to have poor repeatability (precision) and consistently large error in ballast applications.

The nuclear method has two basic variations. The first is a movable device that can be taken to any location for determining density. Either a smooth and flat surface area must be prepared for seating the radiation source and detector, which is impractical in ballast, or the source must be pushed into the material, which is unsuitable because the process of probe penetration disturbs the ballast. With the movable device the large particle size results in unacceptably large error in ballast density. The second variation involves repeated measurements at selected locations either by inserting a nuclear probe (containing source and detector) in an access tube buried in the material, or by burying the probe itself and leaving it for the duration of the test. Again the large ballast particle size results in a potentially large error in density determination. However, the precision may be good enough that changes in density at the same locations may be detectable. The repeatability generally is better with the buried probe because repositioning of the probe in the access tube can cause variation in successive readings.

The special apparatus for ballast density measurement (Ref. 11) was devised by combining past experience with an assessment of the particular requirements for ballast testing. The approach taken

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was a special water replacement method with a membrane lined hole (Fig. 1). The membrane stiffness is important because it controls how much boundary void volume is included in the measured hole volume. The steps in the test are as follows:

1. Prepare a ring of plaster of paris on the ballast surface as a base for the apparatus and as a means of stabilizing the hole.
2. Install the apparatus without removing ballast, and measure the volume of water filling the apparatus and ballast surface voids.
3. Remove the apparatus and then carefully excavate the hole by hand.
4. Install the apparatus and measure the volume of water filling the apparatus and hole including boundary voids.
5. In the laboratory measure the volume of the removed particles and their dry weight.
6. By sieve test determine particle size distribution if desired.
7. Calculate dry density, void ratio, porosity and particle specific gravity.

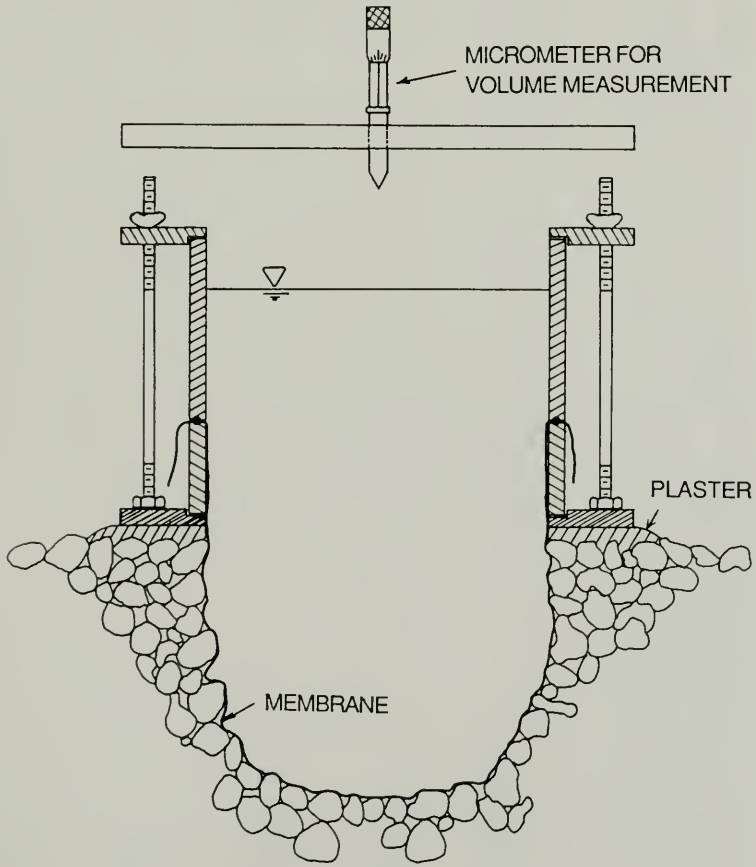

Fig. 1 Ballast Density Test

Field Results

The special apparatus was developed as part of a research program to evaluate ballast compaction in track. Field measurements were made with this apparatus at a variety of railroad sites at locations in the cribs and beneath the ties. The compaction situations included after tamping, after crib surface compaction, and after varying amounts of traffic. The details of this field work are given in Refs. 1, 6, 8, and 9. The apparatus was also used in a research program to evaluate the performance of concrete tie track. Again measurements were made in the cribs and under the ties, before and after maintenance tamping. The details of this field work are given in Refs. 2-5.

For the benefit of the railroad industry, available field data obtained with the special apparatus have been compiled and tabulated in Tables 1-6. The information includes ballast type, location in track, gradation, dry density, void ratio, porosity, specific gravity and the situation defining the state of compaction. A wide range of conditions is represented including a mean particle size from 3 to 41 mm, a uniformity coefficient (D60/D10) from 1.4 to 26, a dry density from 85 to 146 pcf, a porosity from 9 to 45% and a specific gravity from 2.25 to 3.43.

Table 1

SUMMARY OF TEST RESULTS AT FAST DURING INITIAL TESTS

Ref# (Page#)	Ballast Type	Location	Gradation						Results*				Situation
			D90 (mm)	D60 (mm)	D50 (mm)	D10 (mm)	D60/D10	%finer # 4	DD (pcf)	e	n (%)	G	
Ref. 1 (pp. 55) Ref. 9 (Figs.B-1 to B-4)	Granite	Crib Outside Rail	32.0	26.0	22.0	11.0	2.36	0.0	99.2	0.69	41	2.69	After track construction but before traffic except for work trains (extent of work train traffic is not known).
		Inside Rail	23.0	18.0	17.0	11.0	1.64	0.0	101.3	0.64	40	2.66	
		Center	30.0	26.0	23.0	12.0	2.17	0.0	101.0	0.67	40	2.70	
		Under Tie Outside Rail	30.0	24.0	21.0	11.0	2.18	0.0	107.7	0.59	37	2.74	
		Inside Rail	33.0	24.0	22.0	15.0	1.60	0.0	105.1	0.62	38	2.73	
		Center	31.0	26.0	24.0	12.0	2.17	0.0	94.1	0.77	44	2.67	
Ref. 1 (pp. 55) Ref. 9 (Figs. B-5 to B-8)	Limestone	Crib Outside Rail	33.0	27.0	26.0	16.0	1.69	0.0	101.8	0.69	41	2.75	After track construction but before traffic except for work trains (extent of work train traffic is not known).
		Inside Rail	31.0	27.0	26.0	12.0	2.25	0.0	106.3	0.60	38	2.72	
		Center	33.0	27.0	26.0	13.0	2.08	0.0	104.9	0.62	38	2.72	
		Under Tie Outside Rail	31.0	26.0	25.0	15.0	1.73	0.0	112.9	0.53	35	2.77	
		Inside Rail	31.0	25.0	23.0	13.0	1.92	0.0	112.5	0.51	34	2.72	
		Center	31.0	27.0	25.0	14.0	1.93	0.0	98.0	0.76	43	2.76	
Ref. 1 (pp. 55) Ref. 9 (Figs. B-9 to B-12)	Traprock	Crib Outside Rail	46.0	34.0	32.0	20.0	1.70	0.0	111.2	0.66	40	2.96	After track construction but before traffic except for work trains (extent of work train traffic is not known).
		Inside Rail	41.0	33.0	30.0	18.0	1.83	0.0	109.2	0.65	39	2.89	
		Center	37.0	30.0	27.0	17.0	1.76	0.0	107.6	0.68	40	2.90	
		Under Tie Outside Rail	39.0	31.0	29.0	17.0	1.82	0.0	118.7	0.53	35	2.91	
		Inside Rail	39.0	31.0	29.0	17.0	1.82	0.0	117.9	0.55	35	2.93	
		Center	39.0	31.0	30.0	17.0	1.82	0.0	104.0	0.77	44	2.95	

* : Note that Specific Gravity, G, is computed from measured values of dry density and void ratio.

Ref. 1: Panuccio, C.M., Yoo, T.S., and Selig, E.T., "Mechanics of Ballast Compaction, Volume 3 - Field Test Results for Ballast Physical State Measurement", US DOT Final Report No. FRA/ORD-81/16.3, DOT-TSC-FRA-81-3, March 1982.

Ref. 9: Yoo, T.S., and Chen, H.W., "FAST In-situ Ballast Density Measurements", Internal Report, Report No. TSC77-204I, Department of Civil Engineering, SUNYAB, February, 1977.

Table 2

SUMMARY OF TEST RESULTS DURING SUPPLEMENTAL TESTS IN SECTION 20B AT FAST

Ref# (Page#)	Ballast Type	Location	Gradation						Results*				Situation
			D90 (mm)	D60 (mm)	D50 (mm)	D10 (mm)	D60/D10	%finer # 4	DD (pcf)	e	n (%)	G	
Ref. 1 (pp. 59, 65-68)	Limestone	Crib Outside Rail	32.0	25.0	24.0	13.0	1.92	0.00	113.1	0.50	33	2.72	
		Inside Rail	32.0	25.0	24.0	13.0	1.92	0.00	112.6	0.52	34	2.74	
		Center	30.0	23.5	22.5	10.0	2.36	0.00	112.2	0.53	35	2.74	
		Under Tie Outside Rail	30.0	25.0	24.0	13.0	1.92	0.00	114.0	0.50	35	2.74	After 134.6 MGT Traffic
		Inside Rail	30.0	25.0	24.0	13.0	1.92	0.00	114.8	0.48	32	2.72	
		Center	30.0	24.0	23.0	13.0	1.85	0.00	109.7	0.55	35	2.72	
Ref. 1 (pp. 59, 65-68)	Limestone	Crib Outside Rail	32.0	25.0	24.0	15.0	1.67	0.00	98.9	0.73	42	2.74	
		Inside Rail	32.0	25.0	24.0	15.0	1.67	0.00	94.3	0.82	45	2.75	
		Center	29.0	23.5	22.5	11.5	2.04	0.00	110.6	0.55	35	2.75	Tamped After Traffic
		Under Tie Outside Rail	32.0	25.0	24.0	13.0	1.92	0.00	114.4	0.49	33	2.73	
		Inside Rail	32.0	25.0	24.0	13.0	1.92	0.00	113.7	0.50	33	2.73	
		Center	30.0	24.0	23.0	13.0	1.85	0.00	107.6	0.57	36	2.71	
Ref. 1 (pp. 59, 65-68)	Limestone	Crib Outside Rail	30.0	25.0	24.0	15.0	1.67	0.00	112.5	0.51	34	2.72	
		Inside Rail	30.0	25.0	24.0	15.0	1.67	0.00	108.5	0.58	37	2.75	
		Center	28.0	23.0	22.0	11.5	2.04	0.00	112.3	0.52	34	2.74	0.1 MGT of Additional
		Under Tie Outside Rail	32.0	25.0	24.0	13.0	1.92	0.00	112.1	0.52	34	2.73	Traffic After Tamping
		Inside Rail	32.0	25.0	24.0	13.0	1.92	0.00	112.4	0.52	34	2.74	
		Center	30.0	24.0	22.5	12.0	2.00	2.00	106.4	0.61	38	2.75	
Ref. 1 (pp. 60, 69-72)	Traprock	Crib Outside Rail	38.0	30.0	28.0	18.5	1.62	0.00	117.3	0.60	38	3.01	
		Inside Rail	38.0	30.0	28.0	18.5	1.62	0.00	119.2	0.53	35	2.92	
		Center	40.5	30.5	29.0	18.0	1.69	0.00	118.2	0.52	34	2.88	After 134.6 MGT Traffic
		Under Tie Outside Rail	39.0	30.0	28.0	18.0	1.67	0.00	126.6	0.46	32	2.96	
		Inside Rail	39.0	30.0	28.0	18.0	1.67	0.00	127.8	0.45	31	2.97	
		Center	36.0	29.0	27.0	15.5	1.87	0.00	117.1	0.57	36	2.95	
Ref. 1 (pp. 60, 69-72)	Traprock	Crib Outside Rail	39.0	32.0	30.0	21.0	1.52	0.00	119.2	0.55	35	2.96	
		Inside Rail	39.0	32.0	30.0	21.0	1.52	0.00	112.5	0.63	39	2.94	
		Center	36.0	28.0	26.0	15.0	1.87	0.00	117.9	0.55	35	2.91	Tamped After Traffic
		Under Tie Outside Rail	36.0	27.0	25.0	14.0	1.93	0.00	125.8	0.47	32	2.96	
		Inside Rail	36.0	27.0	25.0	14.0	1.93	0.00	113.7	0.62	38	2.95	
		Center	38.0	29.0	27.0	17.0	1.70	0.00	114.6	0.62	38	2.97	
Ref. 1 (pp. 60, 69-72)	Traprock	Crib Outside Rail	38.0	32.0	30.0	18.5	1.73	0.00	118.5	0.55	35	2.94	
		Inside Rail	38.0	32.0	30.0	18.5	1.73	0.00	107.7	0.70	41	2.93	
		Center	36.0	28.0	26.0	15.0	1.87	0.00	120.1	0.52	34	2.93	0.1 MGT of Additional
		Under Tie Outside Rail	37.5	30.0	28.0	16.5	1.82	0.00	124.7	0.48	32	2.96	Traffic After Tamping
		Inside Rail	37.5	30.0	28.0	16.5	1.82	0.00	114.5	0.61	38	2.95	
		Center	38.0	29.0	27.0	19.0	1.53	0.00	115.3	0.58	37	2.92	

* : Note that Specific Gravity, G, has been computed from measured values of dry density and void ratio.

Ref. 1: Panuccio, C.M., Yoo, T.S., and Selig, E.T., "Mechanics of Ballast Compaction, Volume 3 - Field Test Results for Ballast Physical State Measurement", US DOT Final Report No. FRA/ORD-81/16.3, DOT-TSC-FRA-81-3, March 1982.

Table 3

SUMMARY OF CNR TEST RESULTS

Ref# (Page#)	Ballast Type	Location	Gradation						Results*				Situation
			D90 (mm)	D60 (mm)	D50 (mm)	D10 (mm)	D60/D10	%finer # 4	DD (pcf)	e	n (%)	G	
Ref. 1 (pp. 37) Ref. 6 (pp. 26-35)	Nickel Slag	Near Rail	28.5	14.5	12.5	2.6	5.58	15.5	143.6	0.49	33	3.43	Undisturbed Before Tamping
			38.5	18.0	15.5	3.0	6.00	13.5	143.1	0.50	33	3.43	After Tamping
			28.5	16.3	13.6	3.2	5.10	18.6	146.2	0.48	32	3.43	After Tamping and Compaction

* : Because of excessive amount of fines, the void ratio measurements were not made for CNR test sites.
Note that void ratio, e, is computed from the measured values of dry density and bulk specific gravity G = 3.43 (Ref. 7).

Ref. 1: Panuccio, C.M., Yoo, T.S., and Selig, E.T., "Mechanics of Ballast Compaction, Volume 3 - Field Test Results for Ballast Physical State Measurement", US DOT Final Report No. FRA/ORD-81/16.3, DOT-TSC-FRA-81-3, March 1982.

Ref. 6: Yoo, T.S., and Chen, H.W., "In-situ Ballast Density Measurements at Canadian National and Southern Railway Sites", Field Test Report, Report No. TSC76-202I, Department of Civil Engineering, SUNYAB, November 1976.

Ref. 7: Gaskin, P.N., and Raymond, G., "Contribution to Selection of Railroad Ballast", Transportation Engineering Division Journal, ASCE, Volume 102, No. TE2, May 1976, pp. 377-394.

Table 4

SUMMARY OF SOUTHERN RAILWAY TEST RESULTS

Ref# (Page#)	Ballast Type	Location	Gradation						Results*				Situation
			D90 (mm)	D60 (mm)	D50 (mm)	D10 (mm)	D60/D10	%finer # 4	DD (pcf)	e	n (%)	G	
Ref.1 (pp. 40) Ref. 6 (pp. 38-44)	Granite	Crib Outside Rail	40.0	31.5	31.0	22.5	1.40	0.00	109.9	0.59	37	2.80	After Tamping
		Inside Rail	40.0	33.5	32.0	22.5	1.49	0.00	104.8	0.66	40	2.79	
		Average	40.0	32.5	31.5	22.5	1.45	0.00	107.3	0.63	39	2.80	
	Granite	Crib Outside Rail	40.0	35.0	32.5	20.5	1.71	0.00	140.3	0.23	19	2.77	After Tamping and Compaction
		Inside Rail	39.5	32.0	30.5	19.5	1.84	0.00	107.4	0.60	37	2.75	
		Average	39.8	33.5	31.5	20.0	1.68	0.00	123.8	0.42	30	2.82	

* : Note that Specific Gravity, G, is computed from measured values of dry density and void ratio.

Ref. 1: Panuccio, C.M., Yoo, T.S., and Selig, E.T., "Mechanics of Ballast Compaction, Volume 3 - Field Test Results for Ballast Physical State Measurement", US DOT Final Report No. FRA/ORD-81/16.3, DOT-TSC-FRA-81-3, March 1982.

Ref. 6: Yoo, T.S., and Chen, H.M., "In-situ Ballast Density Measurements at Canadian National and Southern Railway Sites", Field Test Report, Report No. TSC76-202I, Department of Civil Engineering, SUNYAB, November 1976.

Table 5

SUMMARY OF TEST RESULTS FROM ICG SITE DURING INITIAL TESTS

Ref# (Page#)	Ballast Type	Location	Gradation						Results*				Situation
			D90 (mm)	D60 (mm)	D50 (mm)	D10 (mm)	D60/D10	%finer # 4	DD (pcf)	e	n (%)	G	
Ref. 1 (pp. 43) Ref. 8 (Figs. B-1 to B-8)	Limestone	Crib Center	37.0	29.0	28.0	17.5	1.66	0.00	96.2	0.60	38	2.47	After Tamping
		Under Tie Center	34.0	28.5	27.5	18.0	1.58	0.00	84.9	0.77	43	2.41	
		Crib Center	35.0	28.0	27.0	20.5	1.37	0.00	91.4	0.69	41	2.48	After Tamping and Compaction
		Under Tie Center	35.0	28.5	27.0	19.0	1.50	0.00	87.9	0.72	42	2.42	

* : Note that Specific Gravity, G, is computed from measured values of dry density and void ratio.

Ref. 1: Panuccio, C.M., Yoo, T.S., and Selig, E.T., "Mechanics of Ballast Compaction, Volume 3 - Field Test Results for Ballast Physical State Measurement", US DOT Final Report No. FRA/ORD-81/16.3, DOT-TSC-FRA-81-3, March 1982.

Ref. 8: Yoo, T.S., "In-situ Ballast Density Measurements at ICG site", Field Test Report, Report No. TSC77-212I, Department of Civil Engineering, SUNYAB, June, 1977.

Table 6

SUMMARY OF TEST RESULTS FROM STREATOR, LORRAINE AND ABERDEEN SITES

Ref# (Page#)	Ballast Type	Location	Gradation						Results*				Situation
			D90 (mm)	D60 (mm)	D50 (mm)	D10 (mm)	D60/D10	%finer # 4	DD (pcf)	e	n (%)	G	
Ref. 2 (Tab 6.1,7.1, 7.2 & Fig. 7.1)	Slag	Crib	38.0	29.0	28.0	11.0	2.6	1.0	91.6	0.53	35	2.25	Before Maintenance
	Slag	Under Tie	32.0	20.5	19.0	3.4	6.0	14.0	116.0	0.26	21	2.34	Before Maintenance
	Sandy Gravel	Sub Ballast	12.0	6.4	5.0	0.6	10.7	60.0	133.0	0.29	22	2.74	Before Maintenance
	Slag	Crib	-	-	-	-	-	-	86.7	0.62	38	2.25	After Maintenance
	Slag	Under Tie	-	-	-	-	-	-	106.0	0.38	28	2.34	After Maintenance
Ref. 3 (Tab 6.1,7.1, 7.2 & Fig. 7.2)	Granite	Crib	40.0	30.0	29.0	20.0	1.5	0.5	107.4	0.62	38	2.78	After Maintenance
	Granite	Under Tie	37.0	30.5	30.0	17.0	1.8	0.5	118.5	0.46	32	2.78	After Maintenance
	Sandy Gravel	Sub Ballast	16.0	4.6	3.4	0.5	10.2	63.0	144.6	0.10	09	2.54	Before Maintenance
Ref. 4 (Tab 6.1,7.1, 7.2 & Fig. 7.3)	Limestone	Crib	40.0	34.0	31.0	15.0	2.3	1.0	102.1	0.60	38	2.62	Before Maintenance
	Limestone	Under Tie	40.0	31.0	30.0	8.0	3.9	6.0	129.6	0.30	23	2.70	Before Maintenance
	Sandy Ballast	Sub Ballast	47.0	30.5	26.0	1.6	19.1	19.0	131.0	0.22	18	2.56	Before Maintenance
	Limestone	Crib	-	-	-	-	-	-	102.2	0.60	38	2.62	After Maintenance
	Limestone	Under Tie	-	-	-	-	-	-	116.2	0.45	31	2.70	After Maintenance
Ref. 5 (Tab 6.1,7.1, 7.2 & Fig. 7.4)	Traprock	Crib	58.0	45.0	41.0	30.0	1.5	1.0	116.1	0.54	35	2.87	Before Maintenance
	Traprock	Under Tie	-	39.0	34.0	14.0	2.8	6.0	135.5	0.32	24	2.87	Before Maintenance
	Sandy Slag	Sub Ballast	-	26.0	23.0	1.0	26.0	20.0	111.7	0.54	35	2.75	Before Maintenance
	Traprock	Crib	-	-	-	-	-	-	103.6	0.73	42	2.87	After Maintenance
	Traprock	Under Tie	-	-	-	-	-	-	135.2	0.32	24	2.87	After Maintenance

* : Note that void ratio, e, is computed from measured values of dry density and bulk specific gravity.

Ref. 2: McMahon, D.R., and Selig, E.T., "Subsurface Investigation Report - Streator, Illinois Wood Tie Site", Department of Civil Engineering, Univ. of Massachusetts, Amherst, Report No. FRA80-254I, November 1980.

Ref. 3: McMahon, D.R., and Selig, E.T., "Subsurface Investigation Report - Streator, Illinois Concrete Tie Site", Department of Civil Engineering, Univ. of Massachusetts, Amherst, Report No. FRA80-255I, November 1980.

Ref. 4: McMahon, D.R., and Selig, E.T., "Subsurface Investigation Report - Lorraine, Virginia Concrete Tie Site", Department of Civil Engineering, Univ. of Massachusetts, Amherst, Report No. FRA80-256I, November 1980.

Ref. 5: McMahon, D.R., and Selig, E.T., "Subsurface Investigation Report - Aberdeen, Maryland, Concrete Tie Site", Department of Civil Engineering, Univ. of Massachusetts, Amherst, Report No. FRA80-257I, November 1980.

Acknowledgement

Rajesh Oberoi, former graduate student at the University of Massachusetts, assisted in compilation of the data.

References

References 1-9 are given as footnotes to Tables 1-6.

10. Selig, E. T., "Maintenance and Traffic Effects on Ballast," Ballast Research, American Railway Engineering Association, Bulletin 678, *Proceedings*, Vol. 81, June-July 1980, pp. 504-520.

11. Yoo, T. S., Chen, H. M., and Selig, E. T., "Railroad Ballast Density Measurement," *Geotechnical Testing Journal*, ASTM, Vol. 1, No. 1, March 1978, pp. 41-54.

12. Selig, E. T., Yoo, T. S., and Panuccio, C. M., "Mechanics of Ballast Compaction, Vol. 2 — Field Methods for Ballast Physical State Measurement," US DOT Final Report No. FRA/ORD-81/16.2, DOT-TSC-FRA-81-3, II, March 1982.

THE DOCKLANDS LIGHT RAILWAY

By: J. Peter Cunliffe, P.E.*

Breaking new ground in the world of transit is the recently constructed Docklands Light Railway of London, England. Most significant is the engineering design, construction, vehicles, and implementation methodology, coupled with an innovative systems approach. These features have together achieved notable success in immediate acceptance, rapidly increasing patronage, and a high demand for capacity enhancement, as well as contributing constructively to commercial development.

Traditional well-tried and tested components (steel wheel-steel rail, electric power, modern control methods) have been melded together with improved technologies, resulting in a new approach to short/medium capacity passenger capability.

The Docklands Light Railway serves parts of London's former up-river docks. Opened on August 31, 1987, the 15 station, 7.5 mile system was built in five years within a budget of $120M,. a strictly cash-limited sum. The railway is operated by Docklands Light Railway Limited, a subsidiary of London Regional Transport (the "Tube"). The initial DLR system consists of three legs joined at a triangular junction. These legs terminate at Tower Gateway, Stratford and Island Gardens. This initial system will be extended by 4-1/2 miles eastward to serve the Royal Docks and Beckton.

The plans for the complete regeneration of the Docklands by the London Docklands Development Corporation identified Light Rail as being the most suitable form of public transport for the area to meet the complicated financial, political, social and time-limited demands placed on the proposed redevelopments.

Many plans were examined to improve the situation, the final answer was found in LRT. The reason was that it is less costly to construct than a conventional rail system, especially as the small, lightweight vehicles can handle steeper gradients and tighter curves than normal trains, requiring less land and reducing the cost of structures.

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*President, Movement Control, Inc.

Docklands Light Railway line and train.

This steel-wheel on steel rail-system is segregated, automatic, high-frequency passenger carrying, normally using automatic operation; much of the route is elevated above ground level.

The word "light" has been used for the system in a similar sense to "Light Rail Transit" or "Light Rail Technology," a worldwide industry that is experiencing growth where conventional heavy rail systems are too expensive and time consuming to build and operate.

A single package contract to design, build and equip the Docklands Light Railway was awarded to a consortium of the two companies GEC and Mowlem, which operated under the name of the GEC Mowlem Railway Group. This was a new and unusual form of contract, which placed the majority of risk with the contractor rather than the employer.

CHOICE OF SYSTEM

A system using conventional standard gage steel wheel on steel rail was chosen for the railway although a number of other systems were considered.

Systems using guided buses were examined but found unsuitable due to traffic congestion, their limited capacity, high maintenance requirement and comparatively short life of the vehicles, potential difficulties with providing electric traction supply, and a lack of space at terminals for turning vehicles.

Rubber-tyred trains were also examined. However, as these require a more complex guidance system than ordinary trains, the extra capital and operating cost involved could not be justified within the cash-limited sum available.

Most of the route could thus follow disused railway alignments, of which a large number existed in the area, providing a base for track foundations and an available right of way while, at the same time, reducing the amount of construction work required.

TRACK

Because of a lightweight rolling stock, the running rails and other track components are of lighter construction than those used on a conventional railway. The rail is 80 lb./yard laid on concrete ties with Pandrol 'e' clip fastenings, continuously welded. In certain areas, concrete slab has been used in place of ties. The maximum gradient is 4.3%, minimum radius 312 feet, 131 feet on specially designed turnouts. Rubber-backed cork pads have been used to minimize noise.

THE VEHICLES

The eleven trains are electrically-powered articulated units of welded steel construction. Two axle single-motor bogies are located at each end of the unit, with a central, no motor two-axle bogie under the articulated joint. Current at 750 volts DC is collected from an outer low-level power rail with bottom contact, allowing the top and sides to be covered for safety. (It is to be noted, however, that the system is also designed for overhead current collection, if needed).

Both rheostatic and disc friction braking equipment is fitted. The former uses the braking effect of electromagnetic forces within the traction motors, automatically compensating for the passenger load, using a load-compensating air suspension system.

Trains are driven by an automatic system and therefore do not have driving cabs. Driving controls at the end of each car are provided to allow manual operations to and from the depot or in the event of a partial system failure.

On each train, interior and exterior displays indicate the destination and two passenger public address systems are fitted. The design includes space for luggage and wheelchairs located towards the center of the car near the articulated joint.

Each unit is provided with automatic couplers so that two or more units can be coupled and run as one longer train when necessary.

The main features of the 2-body articulated, steel construction train are listed below:

Vehicle	Docklands Unit
Length	91 ft., 10 in.
Width	8 ft., 8 in.
Height	11 ft., 2 in.
Weight	36 tons (unladen)
Capacity	84 seats, total 210 persons normal standing, 260 crush load
Maximum service speed	50 mph

The Docklands trains have been provided with sufficient power to achieve a rapid rate of acceleration and to climb the steep gradients on the system. The power-to-weight ratio is 13-3/4 horsepower per ton.

Trains are automatically driven, and conventional trackside signals are not provided. The service is continuously monitored by a central computer which compares the position of trains with stored timetable detail. The central computer communicates with train-board computers when the train is at a station; and depending upon whether the train is running on time, early or late, a stored speed/distance profile is selected.

The on-board computer controls the driving of the trains to the next station stop. A system of Positive Train Identification is used in which each train transmits its identity and destination to lineside equipment to ensure that the correct route is set throughout the journey. Safeguards are provided in the form of a separate failsafe system known as Automatic Train Protection, which prevents trains from exceeding speed limits or collision. The whole system is supervised on a single color coded visual display unit.

The operations maintenance center is located at Poplar, together with facilities for the cleaning and routine maintenance of rolling stock, along with an administrative center, staff facilities, and the control room from where all operations are monitored. A substation at Poplar connected to the National Grid supplies power to the system.

SERVICES

The average scheduled speed, including stops, of trains running from Tower Gateway to Island Gardens is approximately 18-1/2 mph, and from Stratford to Island Gardens is 20 mph, giving total running times of 13-1/2 and 16-1/2 minutes, respectively. About 800,000 train miles are operated annually.

CONCLUSION

The system has played a major part in encouraging new business in the Docklands area. Indeed, its very existence was a prerequisite for the developers of the vast Canary Wharf scheme which is the largest urban development in the world.

During its first year, over 600 businesses came to Docklands. These include a wide variety of companies spanning computers, precision engineering, publishing, food and drink, packaging, as well as a host of small businesses. It is estimated that over 10,000 extra staff have been employed in Docklands as a result—many of whom are doubtless helping to swell the passenger traffic. Ridership increased by 50% to 6.6 million in the first full year of operation.

The author gratefully acknowledges permission given by the Docklands Light Railway Limited to use, and to quote from some of their copyright materials, and appreciates their authority for this article to be published.

RECENT NORTH AMERICAN EXPERIENCE WITH SHELLING IN RAILROAD RAILS (Part II Analytical Treatment)

By: R. K. Steele*

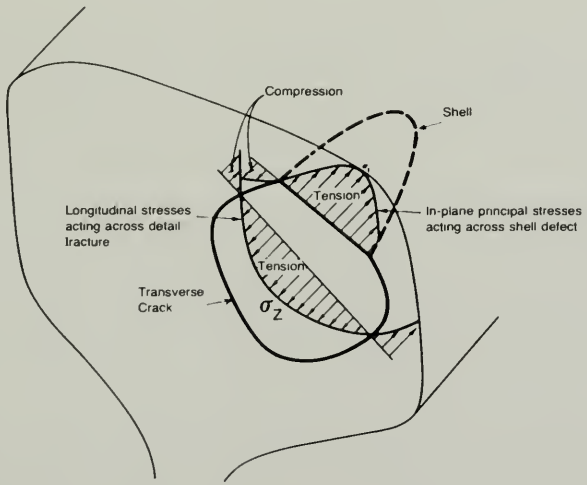
1.0 INTRODUCTION

In Part I of this review (October AREA Bulletin No. 722), the shell was reported most often to initiate internally in close proximity to the boundary between plastically deformed and the undeformed metal under the running surface. Under some circumstances the shell was observed to run horizontally along the rail for hundreds of millimeters without turning downward. But in other cases, the shell could turn transverse to the length of the rail leading ultimately to loss of rail structure integrity.

A sufficient knowledge of the environment in which the shell initiates and grows has developed in the last ten years so that attempts at modelling the shell and transverse defect have been made. Those modelling efforts will be described in some detail along with the predictions of modelling exercises in this second part of the review. The agreement of predictions with observed behavior will be discussed.

2.0 REVIEW

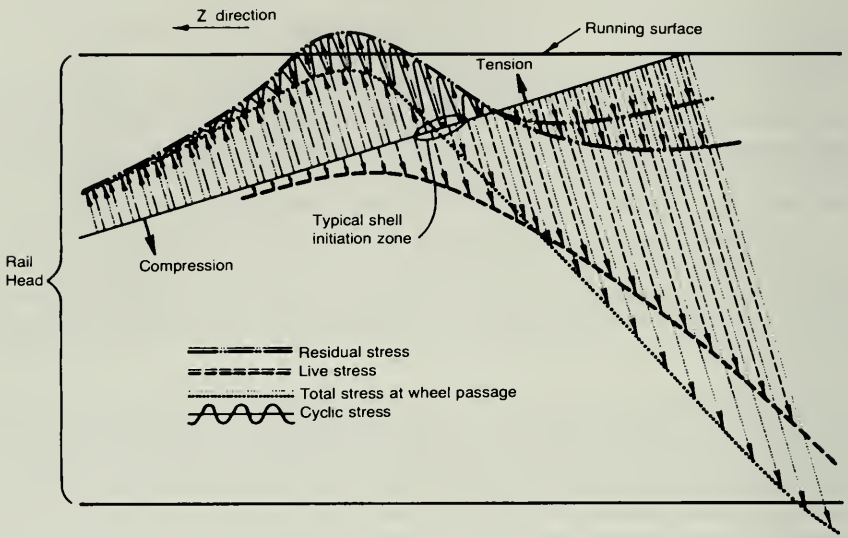
Based upon their observations of shells/TD's and upon their awareness of service-induced residual stresses, Rice et al (1) have proposed a hypothesis for the initiation and growth of the shell; the hypothesis involves the action of both residual stresses and cyclic live contact stresses. Exhibit 1 illustrates the distribution of in-plane (transverse) residual tensile stresses normal to the shell surface and the longitudinal tensile stresses acting upon a transverse crack at a depth well below the shell. The vertical contact stress is shown superimposed upon residual stress acting perpendicular to the shell in Exhibit 2. The maximum cyclic tension stress level occurs at a depth significantly less than that at which the maximum residual tension stress occurs. Although the only cyclic stress illustrated in the exhibit is the tension component, the reader can see easily that the fully reversed cyclic stress (stress ratio $= -1$) occurs at a depth only somewhat more shallow. Both of these depths are close to that at which the shell initiates. Conceptually this hypothesis seems consistent with what has been observed for shell initiation.



Source: Reference 1

Exhibit 1. Diagrammatic Distribution of Residual Stress.

*Director-Metallurgy, Association of American Railroads



Source: Reference 1
Exhibit 2. In-Plane Principal Stresses Shown (Schematically) on an Inclined Plane Down Through the Rail Head.

However, it would not seem easily to explain the wavelike appearance of shell growth under the reversing train type operation at FAST. In revenue service most train reversals occur after only a few train passages; at FAST, reversal has occurred after 100 passages of the 10,000 ton test train. Manifestations of the factors (probably shear stresses) which tend to drive the shell out of plane, while plainly visible on shells formed at FAST, would not be visible easily on shells formed in revenue service. Thus, the initiation hypothesis proposed by Rice et al, utilizing, as it does, only the uniaxial stresses perpendicular to the shell, may not yet tell us all we need to know.

McClintock (2) undertook to examine analytically the behavior of a small horizontal (shell) crack under the influence of repeated load (wheel) passages. "Net tip shear stress" was introduced as a term describing the driving force for crack propagation. Using a boundary integral relaxation method with Hertz equations as boundary conditions along with axial residual stress, he observed that "a wheel passage gives initial sliding, followed by locking, squeezing the plastic zone, reversed sliding, locking, and finally unloading." The analysis included the effects of shakedown, which was achieved in three cylces. Out-of-plane sliding was calculated to be no more than 10% of the in-the-plane sliding of the crack, perhaps as the result of continuous closure of the crack throughout the load passage process. Inclusion of the vertical residual stresses reported by Groom subsequent to McClintock's work might have revealed somewhat greater tendency toward out-of-plane sliding.

More recently Hearle and Johnson (3) have examined possible reasons for the direction effect in contact fatigue crack propagation. Their analysis has assumed "the absence of any significant tensile stress normal to the (shell) surface." Thus they have proceeded from "the view that the cracks propagate in Mode II under the action of shear stresses parallel to the surface" and have utilized the zones of sliding and locking which had been described by McClintock (2). They comment that "the most significant conclusion of this study is to show that the range of Mode II stress intensity factor experienced by the trailing tip of a subsurface crack is greater than that of the leading tip, which

provides a possible mechanism for the observed influence of rolling direction on the propagation of rolling contact fatigue cracks. The directional effects arise thru frictional locking of the crack as it passes under the load. For the effect to become significant, the crack has to be sufficiently long — greater than its depth below the surface — for there to be an appreciable variation in stress from one end of crack to the other, so that parts of the crack can lock while other parts slip." Furthermore with regard to Mode I branch cracks, "it would be expected that such branch cracks would be initiated by high positive values of trailing stress intensity factor, although propagation might be assisted by longitudinal tensile stresses in the rail . . ." This analysis does suggest more rapid growth of the shell in the direction of dominant traffic and allows that the level of the trailing stress intensity can initiate a transverse branch crack. It is not clear whether the tendency to initiate a branch crack is likely to increase monotonically as the shell grows longer.

Farris et al (4) have undertaken an analysis of crack path stability which has included the effects of contact stresses (under a sliding contact) and *both* longitudinal and vertical *tensile residual* stresses. The crack faces were taken to be frictionless. Crack path stability has been defined by the parameter, D_S, such that if it is less than zero the crack is stable, whereas if it is greater than zero, the crack is unstable and will tend to turn out of plane. Exhibit 3 depicts the manner in which the crack path stability parameter is influenced by such factors as the level of longitudinal residual stress (3a), the level of vertical residual stress (3b), the coefficient of running surface friction (3c), and the depth of shell (3d) beneath the running surface. Keeping in mind that values of $D_S > 0$ indicate instability and that a solid line represents downward turning and a dashed line indicates upward turning, the following predictions can be made:

(a) Greater longitudinal residual tensile stress tends to increase the tendency for the shell to turn downward and increases the shell length over which this can occur.

(b) Increased vertical residual tensile stress reduces the tendency for the shell to turn downward and reduces the shell length over which this can occur.

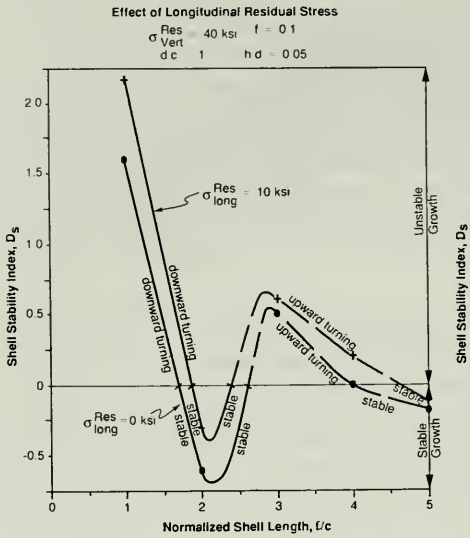
(c) Increased running surface friction increases the length of shell in which downward turning is possible.

and

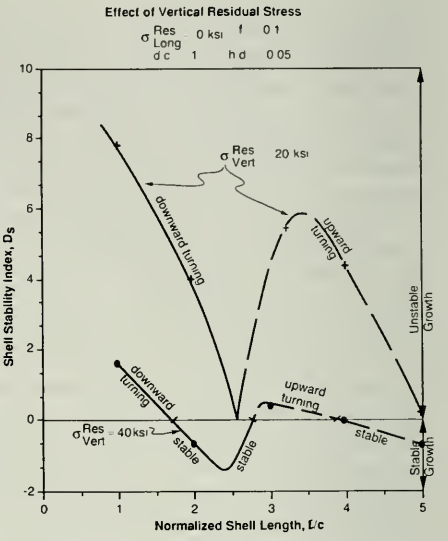
(d) Greater shell depth diminishes the downward turning tendency but may increase very slightly the length of shell over which downward turning is possible.

The extent of out of plane growth can be estimated in the value of the parameter λ. For a fixed value of crack horizontal growth, h, normalized against depth, d, such that $h/d = 0.05$, Exhibit 4 shows that increased longitudinal residual tensile stress and decreased vertical residual tensile stress increase the tendency toward downward turning ($-\lambda$) and increase the length of shell over which such downward turning can take place (larger l/d at $\lambda/d = 0$).

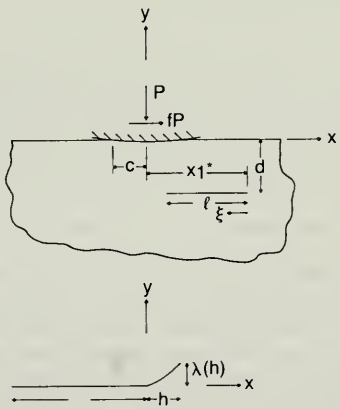
In a subsequent paper by the same authors (5) calculations were made of the crack growth behavior of a shell growing under what are believed to be FAST operating conditions (33 kip wheel load/train reversal each 1 MGT). The predicted growth of the shell (along the rail) is portrayed in Exhibit 5. Both 2D and 3D conditions have been treated in the growth calculation. The point of very rapid acceleration in growth has been predicted to occur between 5 and 10 mm of total shell length in both cases. In the 2D case this condition would be reached in about 10 MGT and in 25-30 MGT for the 3D case. Note that the left side (trailing) portion of the shell grows more rapidly in line with the predictions of Hearle and Johnson (3). The authors propose, following Roberts and Kibler (6), that when the crack gets longer very rapidly, one of the out-of-plane branches becomes too long for the next branch reversal (i.e. train reversal) to negate. At this point, the shell has turned to a transverse defect. Considering the simplifications made in the model (most especially that of zero crack face friction) the prediction is in unexpectedly good agreement with the FAST shell behavior shown in Exhibit 9 of Part I (October AREA Bulletin No. 722). The analytical approach suggests that shells which somehow grow past the critical length for turning down without actually producing a transverse defect (detail fracture) can


Effect of Longitudinal Residual Stress
σ Res Vert = 40 ksi f = 0 1
d c 1 h d = 0 05
σ Res long = 10 ksi
σ Res long = 0 ksi
downward turning
upward turning
stable
Unstable Growth
Stable Growth
Shell Stability Index, Ds
Normalized Shell Length, ℓ/c

3a


Effect of Vertical Residual Stress
σ Res Long = 0 ksi f 0 1
d c 1 h d 0 05
σ Res Vert = 20 ksi
σ Res Vert = 40 ksi
downward turning
upward turning
stable
Unstable Growth
Stable Growth
Shell Stability Index, Ds
Normalized Shell Length, ℓ/c

3b

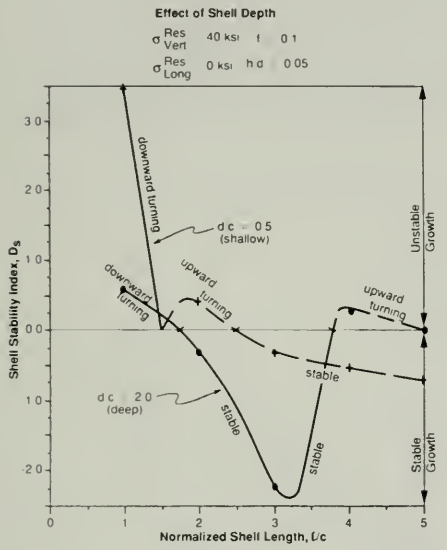


Key for Figures 3 a, b, c and d

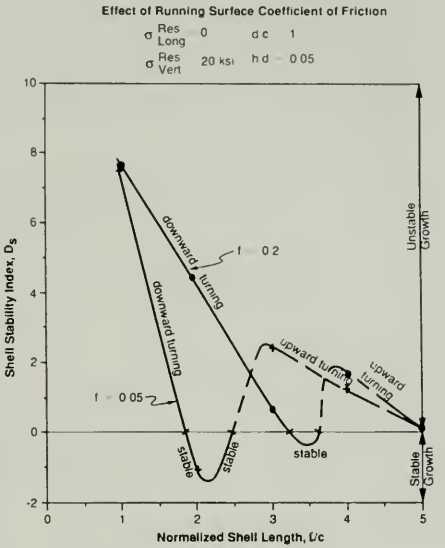
Source: Reference 4

Exhibit 3. Effect of Several Factors on the Crack Instability Factor, D_s.

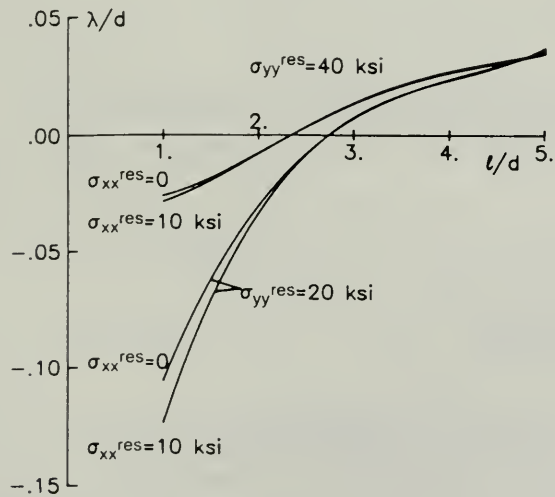
Exhibit 3 Continued


Effect of Shell Depth
σ Res Vert 40 ksi f 0 1
σ Res Long 0 ksi h d 0 05
Shell Stability Index, Ds
Normalized Shell Length, l/c
downward turning
d c = 0 5 (shallow)
upward turning
stable
d c 2 0 (deep)
Unstable Growth
Stable Growth

3c



3d

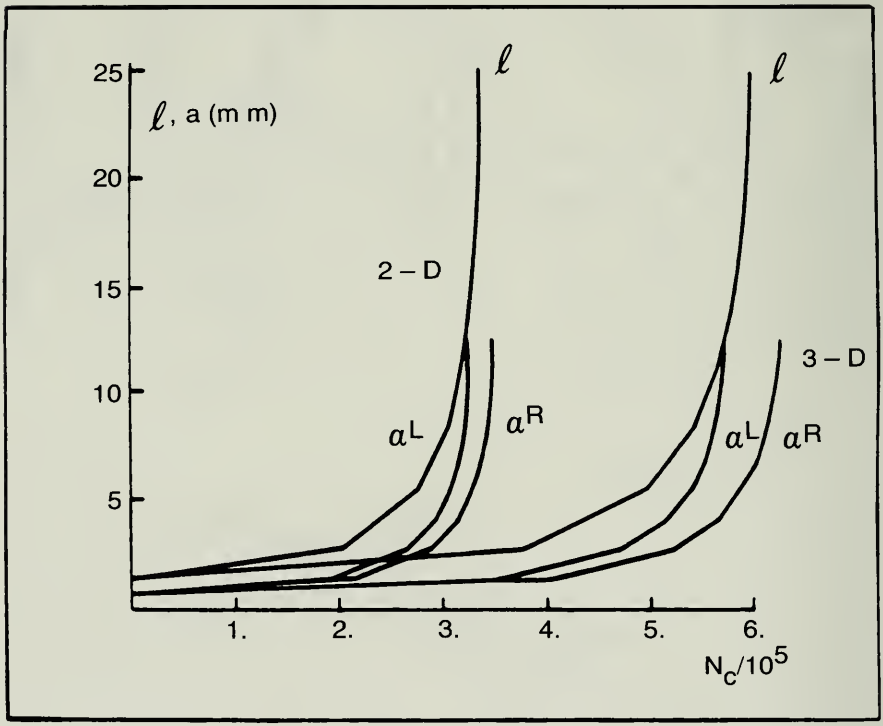


$\lambda(h; l)$ for $h/d = 0.05$, $d/c = 1$, f = 0.1

Source: Reference 4

Exhibit 4. Extent of Out-of-Plane Growth as Function of Normalized Length, λ/d.

continue to grow horizontally without any further tendency to turn downward. Apparently, this is sometimes observed. One may be inclined to wonder how the shell could grow past the critical length for down-turning without actually doing so. A possible means by which this might happen would be for the shell to initiate in the spring of the year, continue to grow thru the critical length in the summer when the axial forces acting along the rail are compressive, and then continue growing along the rail in the fall. If by winter, the shell had not been discovered and it had grown beyond the critical size for the turning downward under winter conditions (i.e. longitudinal tension), it might never develop a transverse component.

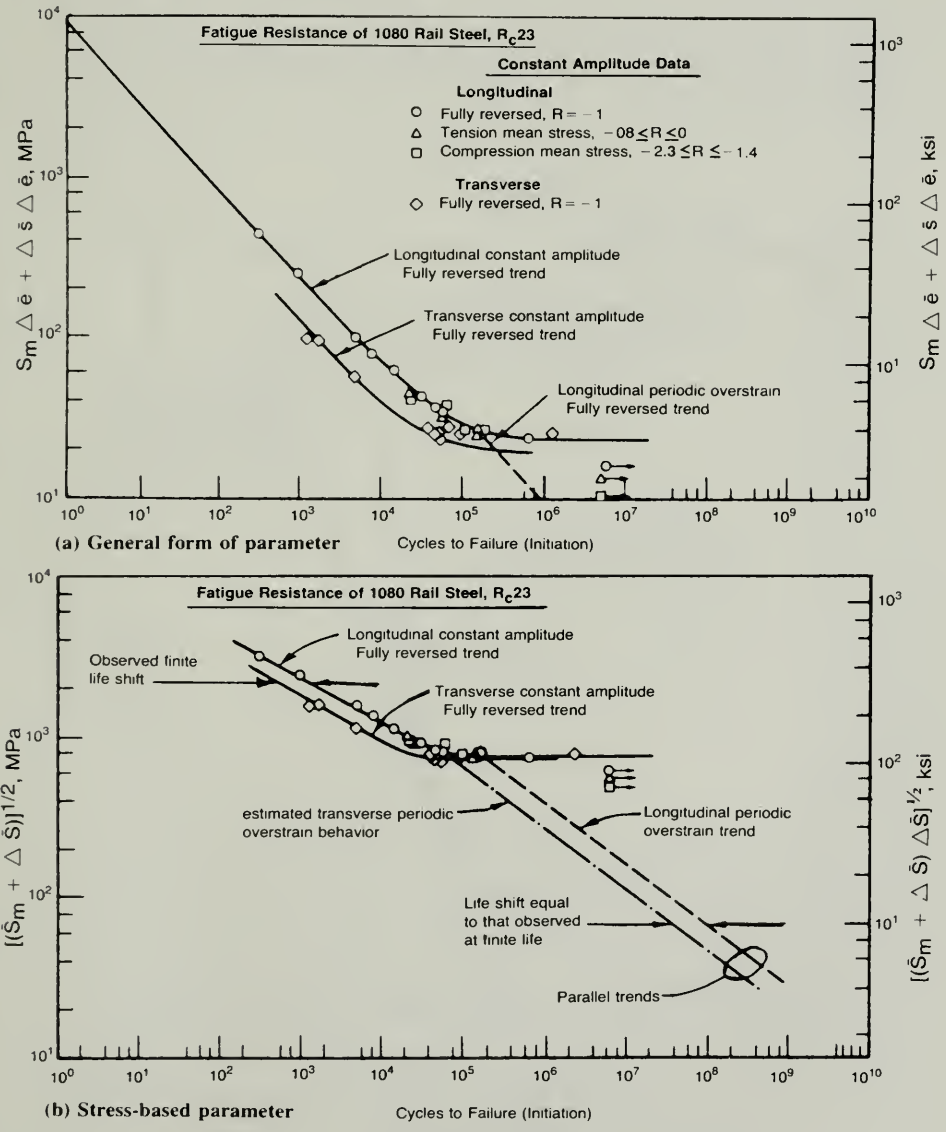


P_o = 1.03 GPa (150 ksi),
Σ^{res}_{yy} = 138 MPa (20 ksi),
f = 0.1,
d = c = 6.99 mm,
$a^L = a^R$ = .699 mm,
ℓ_o = 1.398 mm.

Source: Reference 5
Exhibit 5. Horizontal Shell Length Versus Number of Wheel Passages for Two-Dimensional and Approximate Three-Dimensional Growth.

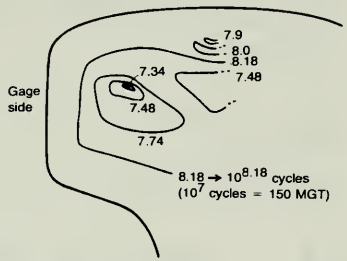
By the early 1980's enough information had been developed about the load environment imposed on rail, residual stresses occurring in rails, and the fatigue and fracture characteristics of rail steel for some venturesome individuals to attempt life calculations for rail head fatigue defects. In North America, Leis and Rice (7) were the first to treat all of the relevant factors including the multiaxial stress fields. Their hypothesis was that "crack initiation is controlled by shear and normal stresses on the material element in the rail head subjected to the greatest combination of thermally induced mean, flow induced residual, and wheel load induced bending and contact stresses." They assessed incurred

fatigue damage using a modification of linear cumulative damage theory coupled with a multiaxial damage parameter calculated from mean stress, stress range and total strain range, the range parameters reflecting the uniaxial equivalents for the multiaxial state. The relationship of life and the damage parameter (in its general form and in an entirely stress based form) is shown in Exhibit 6. They note that periodic overstrains such as may be expected to occur in railroad service can eliminate the occurrence of an endurance limit in rail steels. In the analysis, adjustments to fatigue life have been made for the work hardening which occurs near the surface.


Fatigue Resistance of 1080 Rail Steel, R_c23
Constant Amplitude Data
Longitudinal
Fully reversed, R = −1
Tension mean stress, −0.8 ≤ R ≤ 0
Compression mean stress, −2.3 ≤ R ≤ −1.4
Transverse
Fully reversed, R = −1
Longitudinal constant amplitude Fully reversed trend
Transverse constant amplitude Fully reversed trend
Longitudinal periodic overstrain Fully reversed trend
$S_m \Delta \bar{e} + \Delta \bar{s} \Delta \bar{e}$, MPa
$S_m \Delta \bar{e} + \Delta \bar{s} \Delta \bar{e}$, ksi
Cycles to Failure (Initiation)
(a) General form of parameter
Fatigue Resistance of 1080 Rail Steel, R_c23
Observed finite life shift
Longitudinal constant amplitude Fully reversed trend
Transverse constant amplitude Fully reversed trend
estimated transverse periodic overstrain behavior
Longitudinal periodic overstrain trend
Life shift equal to that observed at finite life
Parallel trends
$[(\bar{S}_m + \Delta \bar{S})\Delta\bar{S}]^{1/2}$, MPa
$[(\bar{S}_m + \Delta \bar{S})\Delta\bar{S}]^{1/2}$, ksi
Cycles to Failure (Initiation)
(b) Stress-based parameter

Source: Reference 7
Exhibit 6. Rail Fatigue Resistance Characterized as a Function of the Damage Parameter.

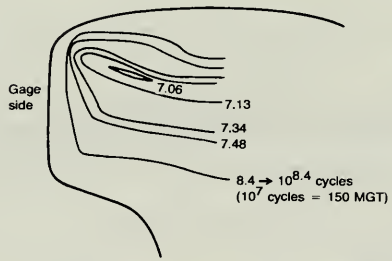
The results of their analysis illustrating the contribution of residual stress are portrayed in Exhibit 7. The shortest life is predicted to occur on the contour identified as 7.06 ($10^{7.06}$cycles —> 172 MGT) under the influence of the in-plane residual stresses. The 7.06 contour zone is not oriented in the same direction as the shell but it is oriented approximately perpendicular to the step which sometimes appears at the origination area of the shell. The depth of initiation calculated (0.4 in. to 0.6 in.) is probably somewhat deeper than would be expected normally for shell initiation. The axial residual stresses also would be predicted to initiate a defect at a slightly longer service exposure ($10^{7.34}$ cycles —>328 MGT); in this case the defect probably would be a detail fracture without any appreciable shell component. Where no residual stresses are present, the life would be predicted to be great ($10^{7.81}$ cycles —>968 MGT) and the maximum damage zone would not be concentrated internally but would extend to the side of the rail head.



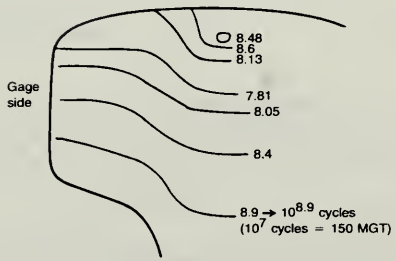
(a) Axial residual stress

Constant Life Contours

134 kN vertical load (30 kip)
Unit train, tangent track
1080 steel, 132 lb/yd rail
In-plane residual stress (83 MGT)
Speed: Slow (static)



(b) Inplane residual stress



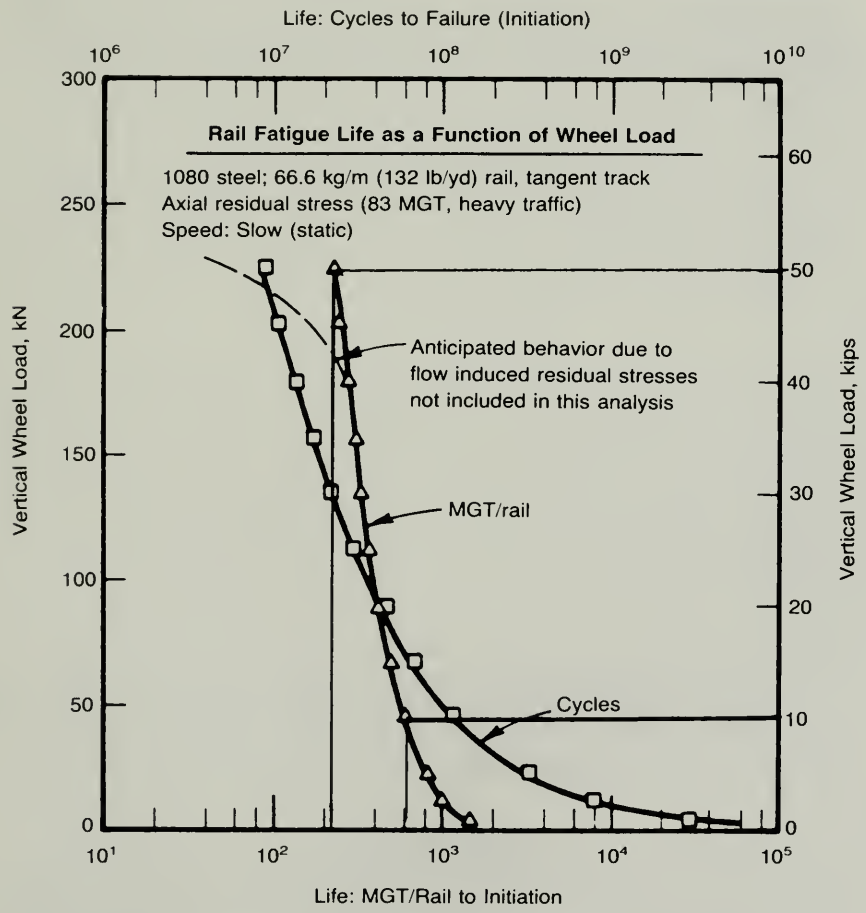
(c) In the absence of residual stress

Source: Reference 7
Exhibit 7. Contours of Constant Damage (life) in the Rail Head — Centered Vertical Load.

The analysis is considerably less clear about the effects of rail section size. With no residual stresses taken into account or by considering only axial residual stresses, the lighter 119 lb/yd section was predicted to exhibit greater service lives than the 132 lb/yd section. Only when different residual stress states were assumed for the two rail sections, did the heavier section have a greater calculated life. The situation with wheel load is somewhat clearer. Exhibit 8 does illustrate a decrease in fatigue life with increased wheel load. But the predicted variation is rather modest; a 5:1 increase in wheel load would be expected to cause only a 3:1 decrease in life measured in MGT. This would suggest that

$$\text{Life (MGT)} \propto (\text{Wheel Load})^{-0.68}$$

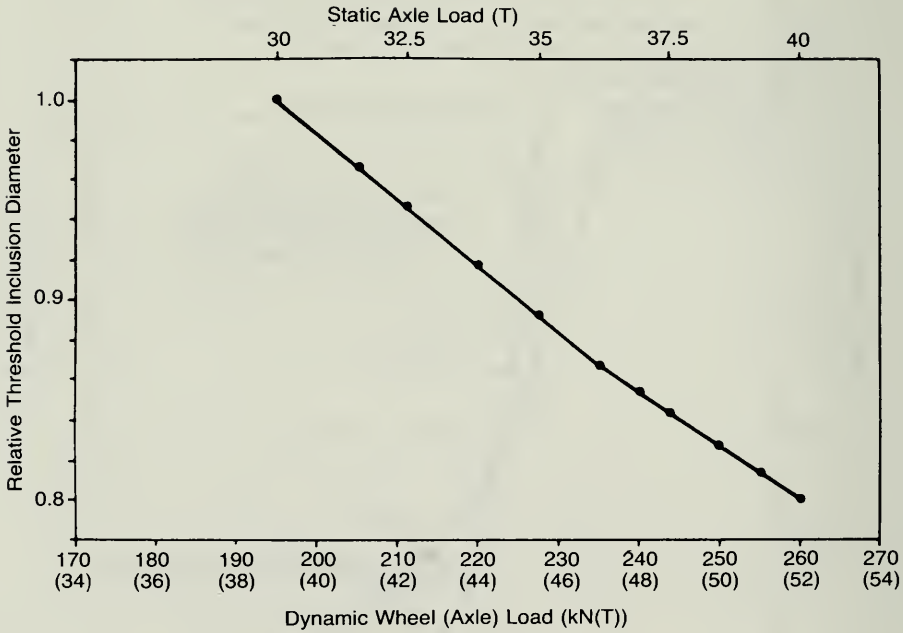
The work of Leis and Rice makes reasonable predictions for initiation of fatigue defects within the rail at approximately the correct depths beneath the work hardened zone. However, the analytical approach does not address the statistical nature of shell formation. As we shall see subsequently, the variation in that statistical nature may be predicted possibly to have large effects on rail service longevity.



Source: Reference 7
Exhibit 8. Rail Fatigue Life as a Function of Wheel Load.

In 1984, Chipperfield and Blickblau (8) described a shell initiation and growth model based upon "available fracture mechanics concepts and guided by extensive prior, metallographic studies of failed rail samples." The failed rail samples came from Australian ore carrying railroad operations where axle loads were comparable to or greater than those encountered in North America. Quite unlike the proposal of Rice et al (1) which puts major emphasis on the vertical (perpendicular to shell surface) uniaxial components of contact and residual stresses, Chipperfield and Blickblau put major emphasis on the shear stresses acting in the plane of the shell, concluding that "bending, residual, and thermal contributions assume lesser importance." Although these authors do not define the residual stress system incorporated into their analysis, perhaps it is relevant to note that their model predicts initiation at 4 mm depth (very close to the depth of maximum octahedral shear without surface traction) whereas actual initiation was observed closer to 10 mm depth. Their initial defect is considered to be the inclusion modeled as a semi-elliptical flaw. They conclude that "contact stresses and inclusion diameter are the major features which determine fatigue response."

Marich and Maas (9) have carried this last view further by noting that to avoid initiation of shells, the maximum allowable inclusion diameter should decrease as axle load increases. This dependency of maximum allowable inclusion diameter on wheel load is illustrated in Exhibit 9. Typically, a 33% increase in wheel load would require a 20% reduction of inclusion diameter.



Source: Reference 9
Exhibit 9. Influence of Load on Relative Threshold Inclusion Diameter.

Others working cooperatively at the Association of American Railroads and the Transportation Systems Center of the U.S. Department of Transportation have adopted an approach based upon the Sines Criterion (10) which allows three dimensional contact, bending, thermal and residual stress states to be treated. Results from the exercise of these models have been presented in references 11, 12, 13, 14. Although the versions of the model developed by each organization differ in detail, each operates by calculating equivalent varying stress amplitudes from the three dimensional stress system. These equivalent stresses then are used to calculate linear cumulative fatigue damage based on laboratory uniaxial S/N data. In the case of the AAR model (PHOENIX), the model accepts an input for work hardening of the rail surface with ensuing improved fatigue properties in that layer. Wear is simulated by the computational removal of surface material while maintaining the work hardening and residual stress gradients fixed with respect to the current surface. The laboratory S/N test data is grouped by life percentile (generally the 1st, 5th, 10th, and 20th percentile are available.) The work of Fowler (15) has served as the source for the S/N data. The residual stresses determined by Schilling and Blake (16) have been used most frequently in AAR studies.

With residual stresses incorporated into the calculation, both versions predict subsurface initiation. The PHOENIX version appears to predict somewhat deeper initiations, generally slightly toward the hydrostatic tension side of the residual stress profile. Perhaps this is related to the inclusion of the effect of surface cold work to increase fatigue resistance near the surface. Joeng et al (12) conclude that rail size, foundation modulus, and wear rate have rather weak effects in the triaxial model, while load and residual stress have only moderate influence. PHOENIX predicts (14) similar behavior. Because the greater longitudinal compression in the head of the rail characteristic of soft track and lighter rail sections is regarded by the model (PHOENIX at least) as a benefit, slightly greater fatigue lives are predicted in these two cases — all other parameters held constant. This appears to be very much like the ORE observations cited in Part I (Reference 24) and perhaps even like those cited by Leis and Rice (7). PHOENIX predicts that the effect of wheel load can be represented by the expression:

$$\text{Lift (MGT)} \propto \text{Wheel Load}^{-m}$$

where m is about 2 if no strong external (thermal) axial forces are imposed but drops to 1.5 if 15 ksi tension is applied along the rail. PHOENIX also predicts little effect of wear rate, at least for low life percentiles (say 1%). However, previously unpublished modelling results show that at greater life percentiles ($\geq 5\%$), greater wear rate appears to have a very beneficial effect in prolonging life (Exhibit 10). This can occur to the extent that the gradual introduction of an aggressive grinding program coupled with a willingness to allow more removal of metal from the ball of the rail ($>25\%$ area loss) could entirely suppress the occurrence of shells at life percentiles approaching 5% or more, (Exhibit 11). Eventually, other rail head defects, most probably the vertical split head and the vertical weld failure would come to dominate the rail failure process. If this prediction is correct, grinding, combined with lubrication, and the use of heavy head rail sections would appear to be able to achieve significant improvements in rail service life.

Encouragingly, improvements in rail life have been reported by Glavin (17), with the ratio of life improvement about the same as predicted by PHOENIX at the specified grinding rate. However, very recent parametric exercises of PHOENIX suggest that the benefit from grinding is very dependent upon wheel/rail contact conditions to the extent that over grinding may change the mode of defect occurrence from shelling to vertical split head formation with an accompanying shortening of life. The occurrence of vertical split heads after implementation of an aggressive grinding program has been observed in revenue service.

An attempt has been made to use some of the life prediction methodology to examine the economic merits of improvements in metallurgical cleanliness (18). If a linear extrapolation of the logarithm of S/N fatigue life were made to "absolute" metallurgical cleanliness, an approximate 100% improvement in life would be predicted. A net present value analysis utilizing this predicted life improvement in PHOENIX has suggested that even a 100% improvement in life is worth a substantial increase in the

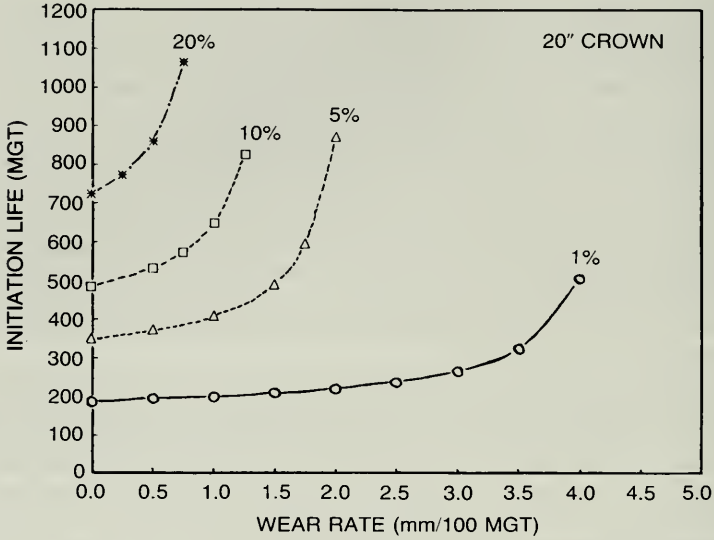


Exhibit 10. Initiation Life vs. Wear Rate.

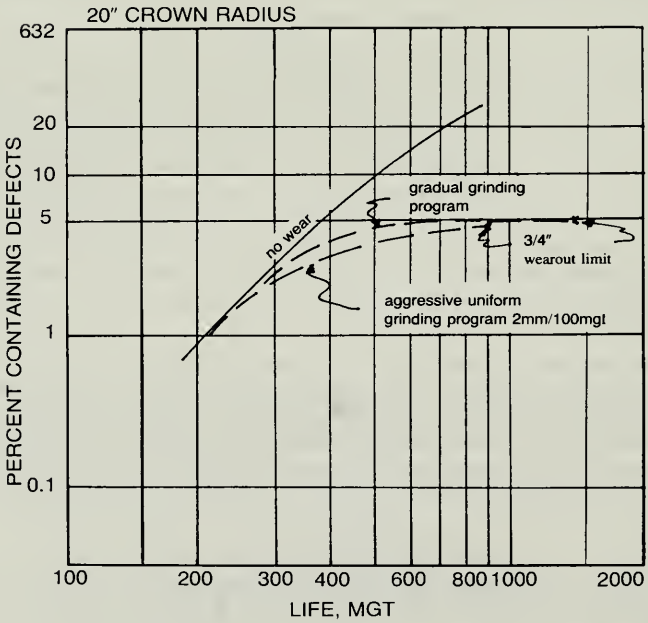


Exhibit 11. Weibull Plot.

price of rail steel. However, the authors point out that the projected life improvement for absolute metallurgical cleanliness can be achieved at far lower premium by strengthening (heat treatment and/or alloying).

The Defect Occurrence and Growth Experiment under way on the FAST High Tonnage Loop has afforded an excellent opportunity to determine whether the models make reasonable predictions about shell/TD behavior at least with regard to rail metallurgical characteristics. Clayton and Brave (19) have analyzed the defect data from the 100T car test phase and find the following:

(1) The shell occurence rate appears to be directly proportional to the logarithm of volume fraction of oxide inclusions as well as some measure of oxide inclusion clustering propensity divided by hardness or perhaps the square of hardness.

(2) For those rails that did exhibit transverse defects (detail fractures) initiating from the shells, there appeared to be an inverse relationship between the defect rate and the logarithm of the product of the two steel cleanliness measures divided by hardness.

(3) Shells longer than 40 mm in length did not produce transverse defects.

The last observation is in good agreement with the predictions of Farris et al. For the first time, in-track test data has shown a linkage between the microstructure cleanliness of the rail steel and the number of shell defects produced. Although direct comparison with fatigue model predictions is difficult, the results of the FAST in-track tests are at least consistent with the model predictions.

3.0 CONCLUDING REMARKS

Based upon the observation of shell defect occurrence in track, fatigue crack initiation and growth models have been developed which provide the engineer with a reasonable means of estimating rail shell defect behavior. The more appropriate of these models make reasonable predictions about the depth of defect initiation, the life to initiation, the turning tendency of the shell with horizontal growth, the effects of non-metallic inclusions, and the interaction of wear (metal removal) and fatigue behavior. What is not so clear is whether the wheel load dependence is predicted appropriately. Nevertheless, the state of model development has progressed far enough to allow satisfactory use for economic costing purposes. Furthermore, exercise of the models suggests several potentially fruitful innovations for extending rail economics life while at the same time they alert us to the dangers that may be inherent in the use of those innovations.

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ERRATA FOR ARTICLE ON "RECENT NORTH AMERICAN EXPERIENCE WITH SHELLING IN RAILROAD RAILS"

AREA Bulletin 722, Volume 90, October 1989

Page 325, third paragraph, change beginning of fourth sentence to "Elongated inclusions at least 400 μ m in length . . ."

PUBLISHED AS INFORMATION BY COMMITTEES

COMMITTEE 5—TRACK

Chairman: W. B. Dwinnell III

EVOLUTION OF THE RAILBOUND MANGANESE FROG

By: E. E. Frank*

During the 19th Century the railroad frog was fabricated from standard carbon steel rail. During this period there were many designs for the rigid frog from riveted plate frogs to the current AREA standard rigid frog.

In the late 1800's R. A. Hadfield of England developed "Hadfield Manganese Steel." The unusual properties of manganese steel, toughness, and its ability to withstand severe impacts made it most suitable for railroad service. The first manganese steel castings were made for street railway frogs. The success of manganese steel in the street railway castings lead to its use in steam railway special work "frogs, crossings and switches." By the 1st decade of the 20th Century the railbound frog was introduced to the American railroads. Since then the railbound manganese frog has progressed thru many design improvements.

Currently there are new designs being developed to meet the needs of the heavy haul railroad.

I. MANGANESE STEEL IN SPECIAL TRACKWORK

The railbound manganese frog evolved from the need to greatly improve the life of the railbuilt frog which was the standard frog during the 19th Century.

During the 19th Century the railbuilt frog was manufactured from Bessemer steel in a variety of designs i.e. riveted plate rigid frogs, clamp type rigid frogs, bolted rigid frogs with cast iron fillers and in later years with rolled steel fillers.

During this era Bessemer railbuilt frogs installed in severe locations would last on the average of three months. The industry recognized that the Bessemer railbuilt frog was a high maintenance-high cost track component, and that a product having both longer life and improved economics was required.

At the present time the railbuilt frog is still being manufactured and installed in accordance with AREA recommended practice in yards and industry tracks where traffic is light.

In the late 1800's R. A. Hadfield of Sheffield, England developed "Hadfield Manganese Steel." The unusual properties of manganese steel, toughness, hardness and ability to withstand severe impacts made it most suitable for railroad service.

The Taylor Iron and Steel Company of High Bridge, New Jersey in cooperation with Sir R. A. Hadfield secured the new development for use in the United States. Hadfield manganese steel was first introduced in the 1890's for car wheels. At the time it was believed that manganese steel would be most suited for car wheels.

However it was not long before it was evident that manganese steel was not suited for this application. When manganese wheels were used in railroad service the wheel tread developed corrugations and excessive flow was experienced during the work hardening period. Shortly thereafter it was learned that manganese steel which was a failure for steam railroad car wheels was a great success for special trackwork over which the car wheels ran.

When the street railways supplanted the horse drawn cars with electric cars, the heavier wheel loads proved to be quite destructive to frogs and crossings then in use. The necessity to improve the designs in the area's of greater wear in track work components became quite evident. It was apparent that the structures would have to be renewable, or durable, or both.

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*Consultant Special Trackwork

The trackwork manufacturers of special trackwork components worked on a solution to this problem. The solution seemed to be a replaceable manganese insert casting known at the time as "Hard Center Work." Designs for the application of a manganese insert casting were developed and a frog with a manganese steel center plate was manufactured and installed on the Atlantic Ave. Railroad in Brooklyn, NY in 1894. The design furnished is shown in Fig. 1. This design utilized lugs cast on the underside of the cast center plate for locking the insert in the frog body.

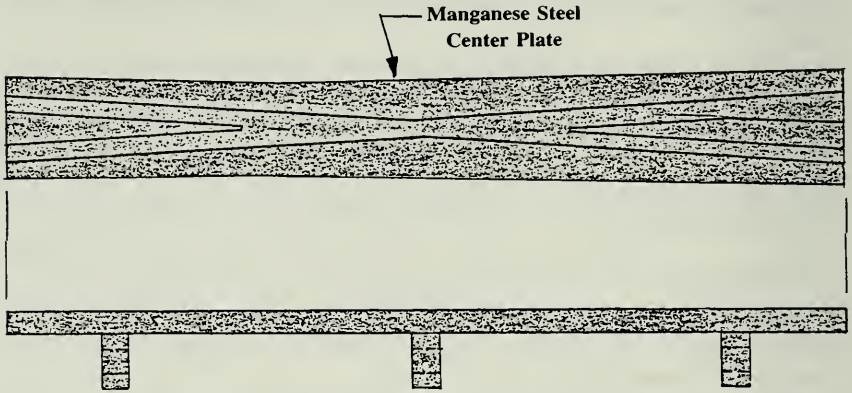

Fig. 1: First manganese steel insert casting used in special trackwork—installed in 1894.

That year several installations were made on electric railways using manganese steel for various trackwork components. The expectations for the superiority of manganese steel for special trackwork castings were more than fulfilled by the results received from these test installations. Shortly thereafter special trackwork components using manganese steel were developed for the electric railways, i.e. Frogs, tongue switches and mates, in both hard center designs and solid construction. The typical hard center frog design used by the electric railways throughout the United States and Canada during the 20th Century is illustrated in Fig. 2.

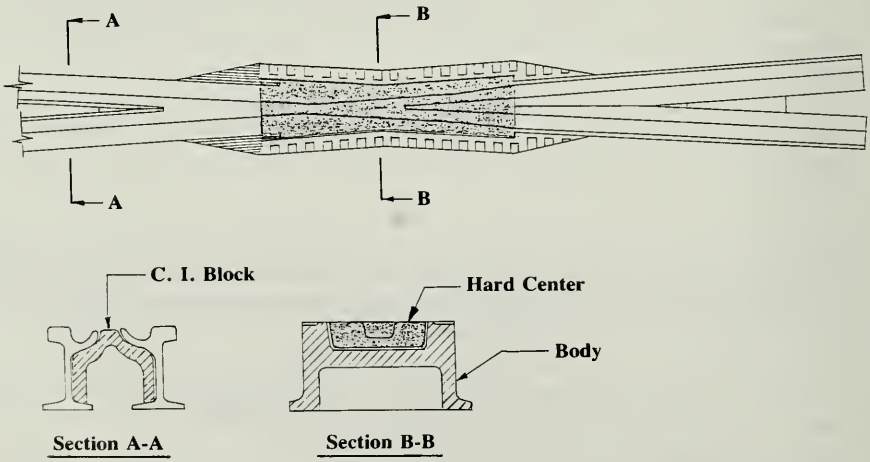

Fig. 2: Typical manganese steel insert design frog used by electric railways used throughout the United States and Canada.

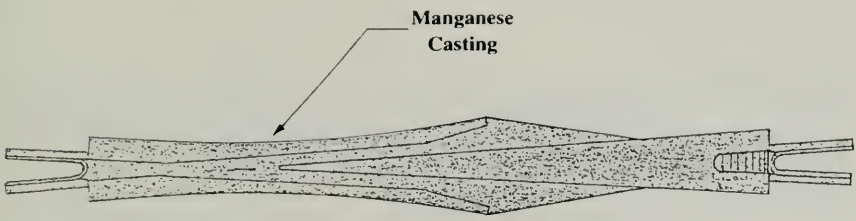

Fig. 3: First solid manganese frog used in special trackwork—installed in 1895.

The first solid construction frog used by the electric railways is illustrated in Fig. 3 and was furnished to the Delaware County Passenger Railway Company of Philadelphia, PA in 1895.

In 1899 the first manganese steel crossing was designed for the Union Traction Company of Philadelphia, PA and is illustrated in Fig. 4. The crossing was to be installed at a location where the electric street railway crossed a steam railway. The solid manganese steel rail of heavy box section was to be for the steam railway track and the Bessemer rail for the electric railway track.

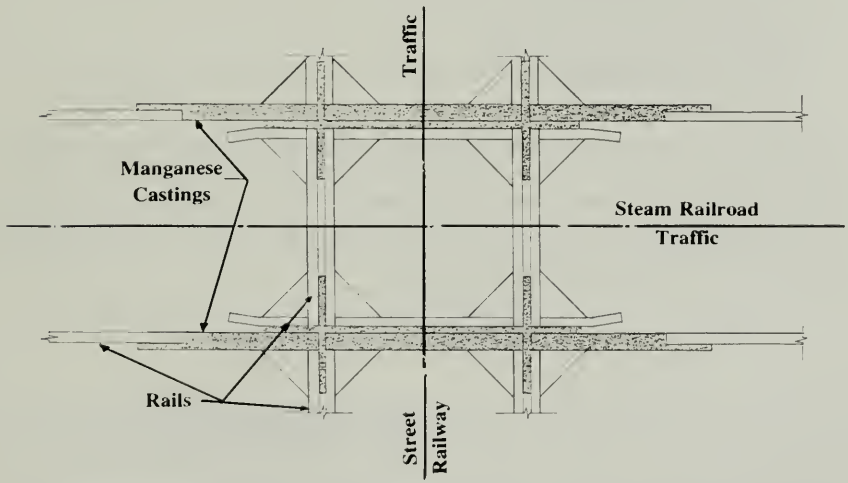

Fig. 4: First manganese steel crossing installed in track. (steam railway crossing electric railway)—installed in 1899.

During the same year a solid manganese frog as illustrated in Fig. 5 was designed for the Pennsylvania Railroad to be installed in Philadelphia, PA, being the first solid manganese frog installed on a steam railroad. The frog was installed in 1900 replacing a Bessemer railbuilt frog. The Bessemer railbuilt frog was lasting on an average of three months whereas the solid manganese frog that replaced it lasted seventeen times as long. The solid manganese frog was removed from track once for regrinding to good surface. The frog was then replaced in track in the same location, finally being removed with a total service life equal to the life of twenty-five Bessemer railbuilt frogs. The results of this test installation's service life did not change the misgivings of the steam railroad engineers. The steam railroad engineers were concerned with the possible breakage of the casting in high speed locations. To overcome this objection and the objection raised relative to the necessarily short length of the solid

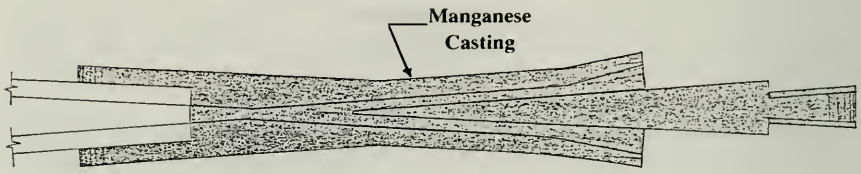

Fig. 5: First solid manganese frog installed in steam railway track—installed in 1900.

manganese frog, the railbound manganese frog was designed (Fig. 6). The first railbound manganese frog was installed in the Baltimore Terminal on the Pennsylvania Railroad in 1900. After two years of successful service the railbound manganese frog gained the confidence of the steam railroad engineers and their use in high speed service was established. During the period from 1900 to 1910 the use of manganese steel was extended to:

—solid manganese crossings
—railbound manganese crossings
—solid manganese guard rails
—manganese steel faced guard rails
—railbound manganese spring frogs
—manganese steel pointed split switches
—cast manganese steel rail
—rolled manganese steel rail

With the use of manganese steel in special trackwork having been firmly established and the economic benefits obtained in severe service widely recognized, the eastern railroads began extensive use of the unique metal for special trackwork components.

The Europeans were closely watching the results being obtained in the United States and when manganese steel was finally introduced in Europe for application to special trackwork it was apparently received with less skepticism than in the United States.

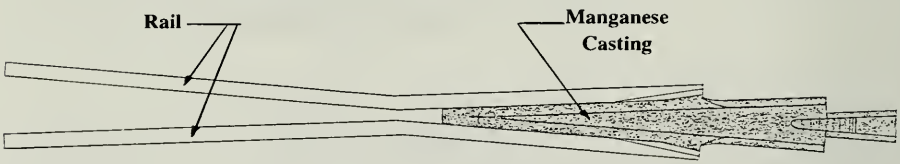

Fig. 6: First railbound manganese steel frog—installed in 1900 on the Pennsylvania R.R.

II. DEVELOPMENT OF THE RAILBOUND MANGANESE FROG

After the successful service of the first railbound manganese frog installation at the Baltimore Terminal in 1900 the railroad engineers gained confidence, and in 1902 a railbound manganese frog was installed in high speed service on the Pennsylvania Railroad.

The success of this installation established the use of manganese steel in special trackwork. The eastern railroads recognized the economic benefits of manganese steel in special trackwork and began extensive use of this unique metal, commonly referred to as "the metal par excellence for the purpose."

During the succeeding years there have been many attempts to develop a metal superior to manganese steel, however to date none has been found.

From 1900 to the 20's there were many designs for special trackwork components developed and tested by the steam railroads, resulting in improved designs and service life to meet the demands of the ever increasing wheel loads and higher speeds.

The first railbound manganese frog design illustrated in Fig. 6 was introduced with modifications by the Pennsylvania Railroad until the 40's. This design was very successful but as the wheel loads increased the short heel length created wear problems resulting in the heel joint becoming loose, increasing maintenance.

A new design railbound manganese frog was introduced by the Ramapo Iron Works in 1905, as illustrated in Fig. 7. There were two basic variations to this design, one as shown in Fig. 7 with extended fillers and the other design without extended fillers.

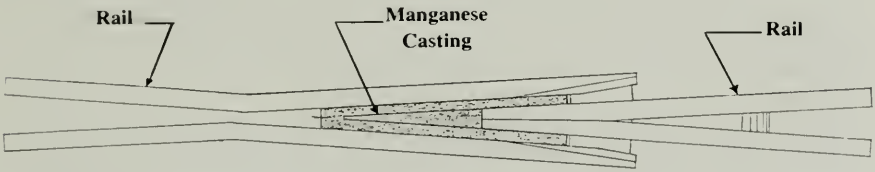

Fig. 7: Typical railbound manganese frog introduced in 1905 and used extensively by steam railroads.

The frog design illustrated in Fig. 7 was used extensively in the new Grand Central Terminal which was being constructed during 1906 to 1911. Other eastern railroads made extensive use of this new design Railbound Manganese Frog. A modification of the design shown in Fig. 7 is still in use today and is illustrated in Fig. 8. Note: the frog illustrated in Fig. 8 has manganese wings which were introduced about 1915.

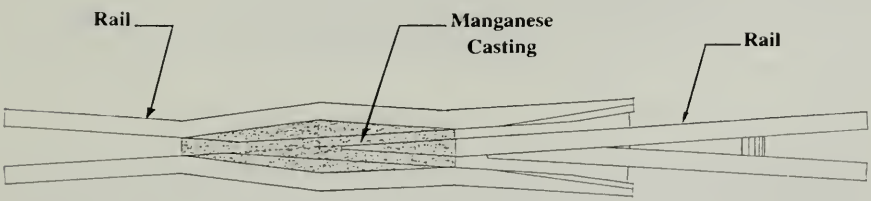

Fig. 8: Frog design currently in use based on the design in Fig. 7.

As the wheel load increased the false flange (in wheel terms hollow tread) developed, causing crushing and wear on the receiving guard line or gage line in the area where the false flange traverses the flangeway.

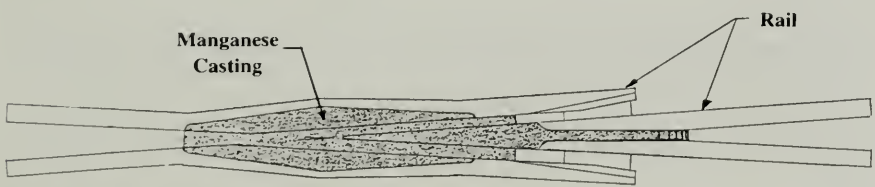

Fig. 9: Improved frog design introduced in 1915.

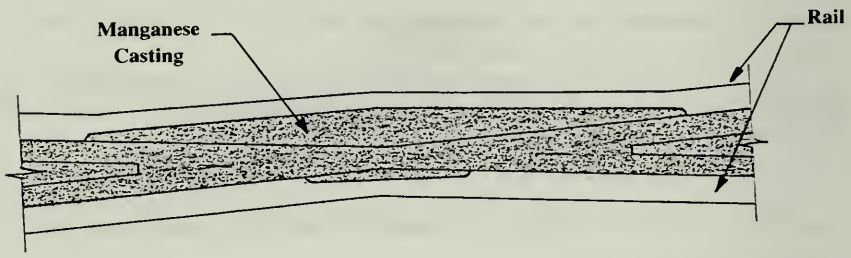

Fig. 10: Application of integrally cast manganese wear surfaces to special trackwork components.

To overcome this problem in 1915 manganese wings were added to the basic design to provide a wearing surface in this area. The manganese provided a surface that work hardened, reducing wear and maintenance. This modified design as shown in Fig. 9 has the manganese wings fit a milled recess in the wing rail. To improve the heel rail connection a heel extension was added to provide a means of attaching the heel rails to the manganese insert casting. The manganese recess at the toe end provided a continuous line on the gage line which was desirable but as wear occurred the manganese flowed resulting in chipping and in some instances breakage of the manganese guard. During this same period integrally cast manganese wear surfaces were added to the railbound manganese center frog casting at the bend in the guard rail as shown in Fig. 10. This wear strip was discontinued in the late teens as new designs became available. By the mid 20's the railbound manganese frog design illustrated in Fig. 11 was developed and became the AREA standard railbound manganese frog referred to as the AREA 600 design.

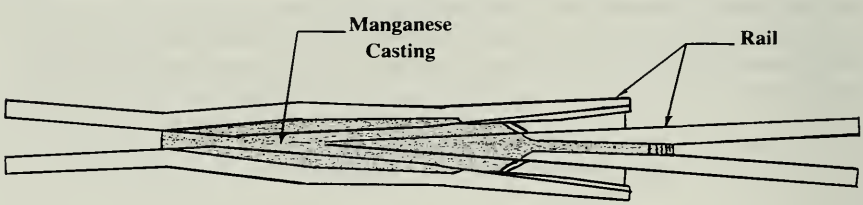

Fig. 11: AREA railbound manganese frog design introduced in the 20's.

This design was in universal use until 1946 when the current AREA 621 design railbound manganese frog illustrated in Fig.12 was introduced. As the wheel loads increased it had become evident that a heavier frog was required. The main deficiency in the 600 design railbound manganese frog was the weak section where the heel extension connected to the body of the frog, resulting in breakage at this location. The new AREA 621 design frog had heavier walls and the section where the heel extension connects to the body of the frog was improved, in addition the notch in the wing rail was eliminated. To improve the 621 design railbound frog and reduce maintenance the depressed heel Fig. 13 was adopted in 1971. The depressed heel permits the wheel load to be carried by the wheel tread on the heel rail and manganese insert and gradually transferring the wheel load from the wheel tread to the false flange when the false flange engages the ramp which is located inside the body of the casting where there is a stronger section. The normal plastic flow and resulting chipping experienced by the original AREA 621 design railbound manganese frog required grinding in the heel extension area to control the metal flow and eliminate chipping. The depressed heel has since been adopted as a standard by AREA for the 621 design (heavy wall) and the 616 design (medium wall) railbound manganese frogs.

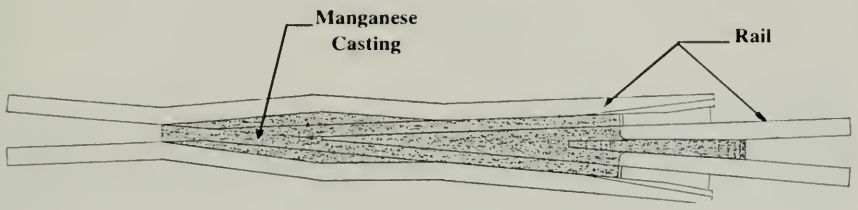

Fig. 12: AREA railbound manganese frog design introduced in 1947.

During this same period the integral base design frog was introduced using the same design criteria as the AREA 600 and AREA 621 design railbound manganese frogs except with the sections as shown in Fig. 15A. This frog design has been used with success in heavy haul locations.

During the last decade the number of heavy haul lines and unit trains consisting of 100 ton cars has greatly increased. This increase in high tonnage cars has developed the need for an improved railbound manganese frog.

In 1980 a new design railbound manganese frog was introduced to meet these demands. This new design railbound manganese frog is illustrated in Fig. 14 and typical sections are illustrated in Figures 15A and 15B.

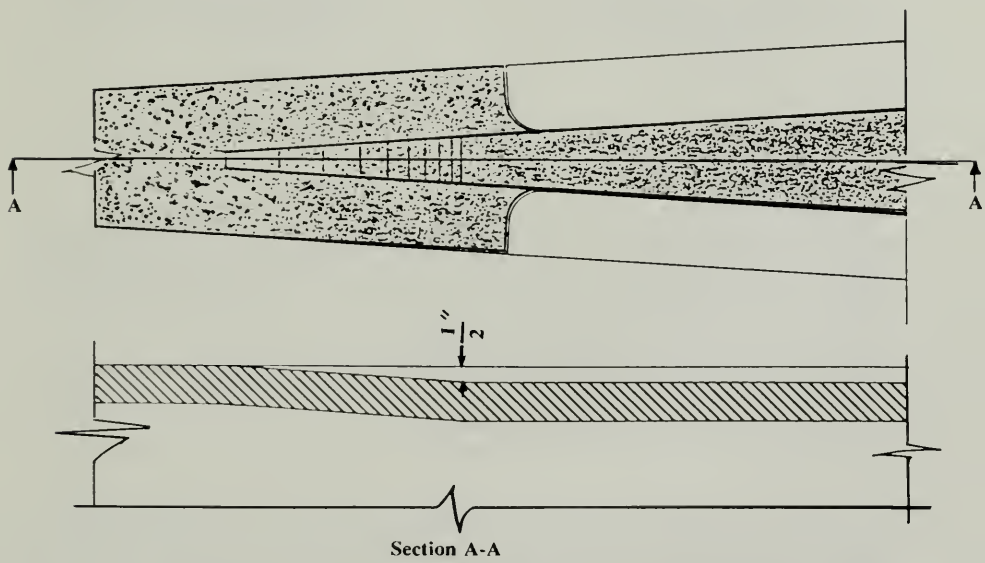

Fig. 13: Depressed heel for AREA 616 & 621 design railbound manganese frogs introduced in 1971.

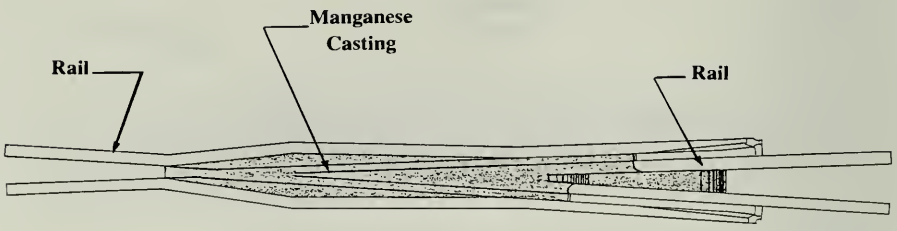

Fig. 14: Improved railbound manganese frog designed for heavy haul railroads introduced in 1980.

To overcome the failure of the heel extension on the current railbound manganese frogs, the new design has the joints where the heel rails connect to the manganese casting staggered, rather than the joints being opposite as the existing design requires. The staggered joints provide improved sections in the heel area that are stronger than the existing design, in addition the casting section is improved.

Fig. 15A

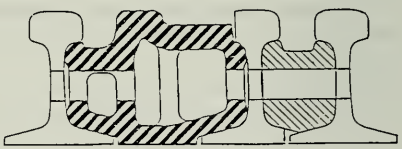

Fig. 15B

Fig. 15A & 15B: Sections for integral base design frogs—Figures 13 & 14.

III. MANGANESE IN EUROPE

The use of manganese steel for special trackwork having been firmly established in the United States was introduced in Europe with great success. The results obtained in the United States with manganese steel in the application of special trackwork had been closely monitored by the Europeans. The Europeans began using solid manganese frogs and crossings and experienced the same results as the United States railroads—longer life and economic returns.

It was reported that one installation on the Central London Line at the British Museum Station was in use fourteen to fifteen years handling approximately 700 mgt. of traffic where as the railbuilt crossings previously had a life of six to eight weeks.

Solid manganese construction is still in general use throughout Europe for special trackwork applications.

A typical solid manganese frog currently in use is illustrated in Fig. 16. It is to be noted that the solid manganese frog illustrated has integrally cast rail sections on each end for attaching the frog to the connecting rails. This connection design has been used in North America for electric railways and industrial applications only and has not been used on mainline railroads because of the difficulty in maintaining tight joints.

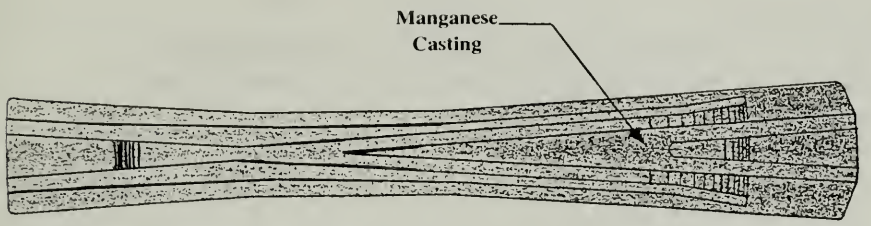

Fig. 16: Typical European Monobloc Frog.

To date the railroad manganese frog has not been adopted by the European railroads undoubtedly because of the difference in track structure and wheel loads. However, the railbound manganese frog of various configurations are in use in South America, Mexico and the heavy haul railroads in Australia. Currently the European railroads are developing all welded frogs and crossings where machined elements are welded together to form a frog or crossing.

IV. CURRENT NEW FROG DEVELOPMENTS

The preponderance of 100 ton cars and unit trains in the last decade have developed the need for frogs that will withstand the impacts delivered as the wheels cross over the flangeway from the point to the guard (or wing) surface or conversely from the guard to the point. Currently frogs have the receiving guard line or gage line crushed by the false flange requiring maintenance.

To improve the dynamics of the turnout the welded/epoxy bonded turnout was designed and tested in 1972 on the Penn-Central Railroad. The tests proved successful, and with the elimination of bolted joints, switch heel joints, and short frog arms, the dynamics of the turnout were greatly improved. The long switch point rail and long frog arms permitted the natural track wave to propogate through the switch and frog providing a smoother ride. The welded turnout also included long guard rails with the guard rail flare opposite the frog flare, this greatly reduced the lateral movement of the frog reducing stresses in the casting and frog bolts.

In 1984 a spring frog for welded/epoxy bonded turnouts was introduced and tested on Amtrak in the Northeast corridor. The tests have been successful providing a continuous surface for the wheel tread to traverse. The long frog arms dampen the vertical movement of the spring wing and provide additional force biasing the spring force. It is to be noted that a spring frog should only be used in a location where 80% or more of the traffic is on the main line and 20% or less is for the turnout run. Further improvements are being sought and a new generation of frogs is being developed, namely moveable wing frogs and moveable point frogs. These designs provide a continuous surface for the wheel tread to traverse thus eliminating the impact delivered by the wheel crossing a flangeway. These frogs are still in the testing stage and results are to be evaluated. The main drawback being economics since an extra machine is required for the frog and the necessary circuitry to have the switch and frog thrown in correspondence.

V. CONCLUSION

Today manganese steel in special trackwork is extensively used throughout the world and still remains "the metal par excellence for the purpose" and since its introduction nothing has been found superior to it.

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1. Practical Trackwork by W. F. Rench.
2. Manganese Steel by Oliver & Boyd.
3. Historically Speaking by K. Taylor.
4. AREA Portfolio of Trackwork Plans.
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COMMITTEE 7 — TIMBER STRUCTURES

Chairman: W. Benton III

Report of Subcommittee D-3-82

Subcommittee Chairman: A. S. Uppal

EFFECT OF DAPPING AND END OVERHANG ON ALLOWABLE STRESSES IN BRIDGE TIES

INTRODUCTION

The horizontal splitting of bridge ties on steel spans was reported by some railroads. The splitting appeared to have occurred either about mid-depth of ties or emanated from an upper inside corner of a dap and progressing away from the dap. Since splitting reduces the load carrying capacity of ties, a few years ago the AREA Committee #7 decided to undertake the study of the effect of dapping and end overhang on the allowable stresses in bridge ties. Consequently a questionnaire was prepared in 1988 and after its approval from the AREA Board was forwarded to twenty class number one railroads. At the time of compilation of this report only fifteen railways had responded. However, one of the respondents did not complete the questionnaire. A summary of the fourteen responses is given in the following:

A. General

1. Of the fourteen respondents, seven indicated to have open deck bridges in majority, six to have ballast deck bridges in majority and only one had decks of other type in majority. The average percentage of each type of deck and their respective ranges were as follows:

Deck Type	Average Percentage	Range Percentage
Open Deck	52.1	11 to 90
Ballast Deck	40.8	04 to 89
Other	7.1	00 to 70

The percentages for the individual respondents are shown in Figure 1.

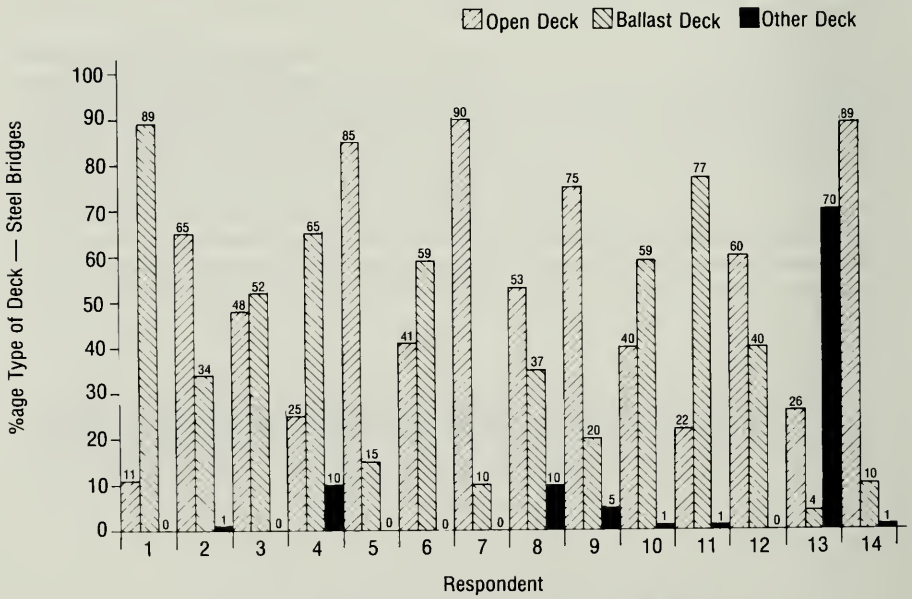

Fig. 1 — Percentage of Deck Types by Respondent

2. Regarding the use of rail anchors on bridges, seven of the fourteen railroads stated they use rail anchors on both the jointed as well as the welded rails, two use only on jointed rails and two only on welded rails. The most commonly used rail anchors were of the drive-on type followed by the spring type. Their spacing varied from box — anchoring every tie, to every other tie, to every bolted tie and depending upon the type and length of span, to as necessary to protect the rails from creeping.

B. Bridge Tie Standard

1. The species of wood used for bridge ties by different railroads in order of preference were pine, fir, oak and gum. Some used other species of hardwood and one of them had started using the imported azobe. Three of the fourteen respondents stated they only used pine, three only fir, while another only oak and remaining a combination of the species. Different grading rules referred to for fir were NLGA, WCLA and WCLIB, for pine SPIB and oak NHLA.

2. For open deck bridges, a majority of the railroads maintained the same width ties for different centre to centre of girders. Six of the fourteen respondents used 10″ wide ties, three used 9″ and another two 8″. The remaining two used a mixture of 8″ and 10″, and 10″ & 12″ width depending on the spacing of the girders. The average width for fourteen respondents was 9.4″ with variation from a min. of 8″ to a max. of 10″. The depth of ties varied with the spacing of girders.

   The gap between ties varied from 4″ to 7″ with an average of 4.64″. Nine out of fourteen used a gap of 4″, two of 5″, another two of 6″ and the remaining one at 7″. This provided a deck coverage of 56% to 75% with an average of 67%.

   On the assumption that the axle load was taken by three consecutive ties as indicated by almost all the respondents, the length of deck over which the dispersion took place varied from 36″ to 48″ with an average value of 42″.

   The depth of ties, their cross-sectional areas and their section moduli are shown in Figures 2, 3 and 4 respectively. The shaded areas show the range of variation and the encircled digits indicate the number of respondents.

3. The size of the tie plates commonly used were double-shouldered 7-3/4″ wide by 14″ or 14-3/4″ long. One railroad used 8-3/4″ wide tie plates and depending on the rail size, curvature and tonnage etc. their lengths of up to 18″ have been employed. Eleven out of fourteen respondents reported using tie pads while three stated they do not use tie pads. The common type of tie pad is 1/4″ neoprene while occasionally corded rubber, preformed fabric, fabreeka "Fabco super seal" or Korvex "Tire Cord" etc. have been used.

C. Dapping of Ties

1. Nine out of the fourteen respondents dapped ties, two occasionally dapped while the remaining three did not dap ties. Six of those who dapped ties, did so to provide deck stability, to prevent lateral movement of deck as well as to obtain proper rail surface. Another two dapped only to obtain proper rail surface.

   In most instances, the depth of dap was 1/2″ and its width was 1/2″on each side of girder/beam flange for tangent track and slightly more for curved track.

2. Two of the three that did not dap their ties, appeared to have the greatest number of open decks. These respondents sized their ties on one side.

3. Superelevation — Eleven out of the fourteen provided superelevation for curved track on bridges, one provided it only where there was no other way whereas two respondents stated they did not provide superelevation. A majority of them tapered ties for superelevation, a few provided by dapping or both by tapering and dapping while a couple of them used timber riser blocks for superelevation.

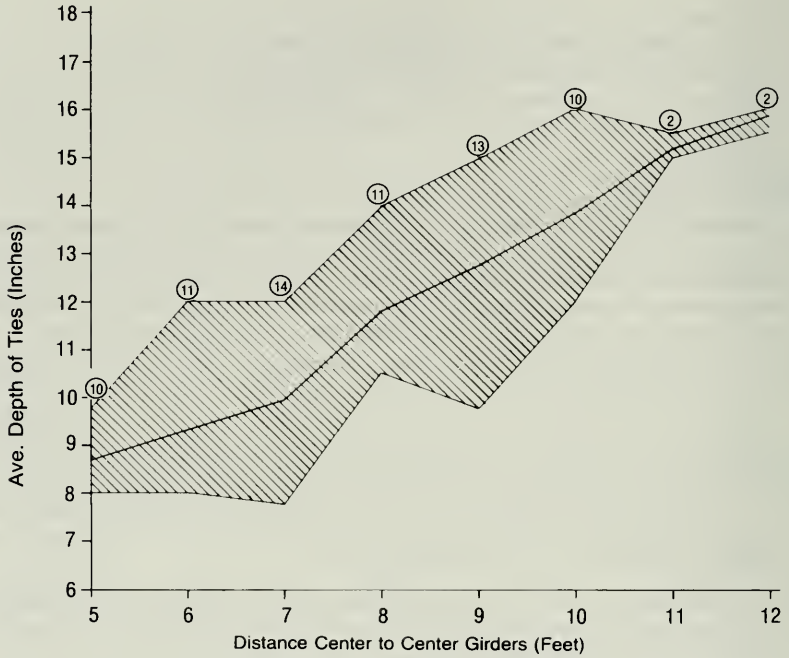

Fig. 2 — Depth of Bridge Ties for Open Decks

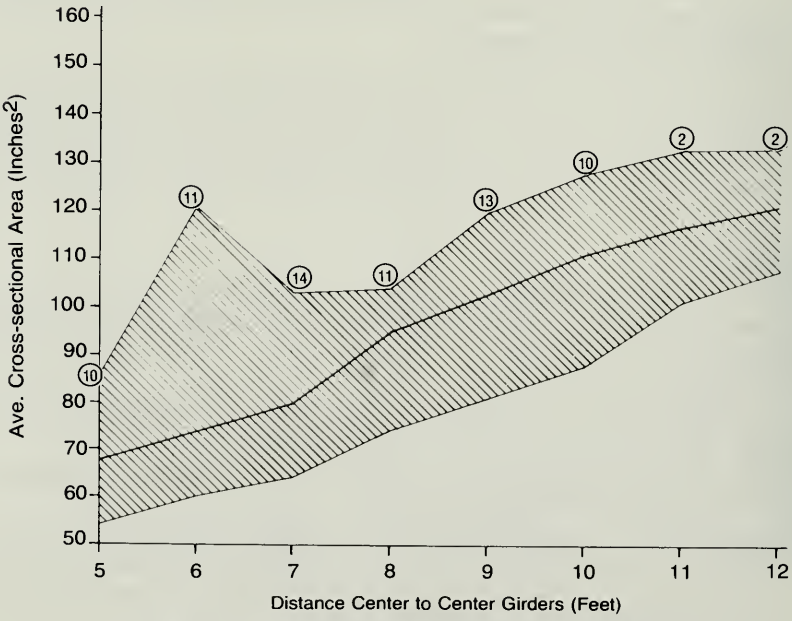

Fig. 3 — Cross-section Area of Bridge Ties for Open Decks

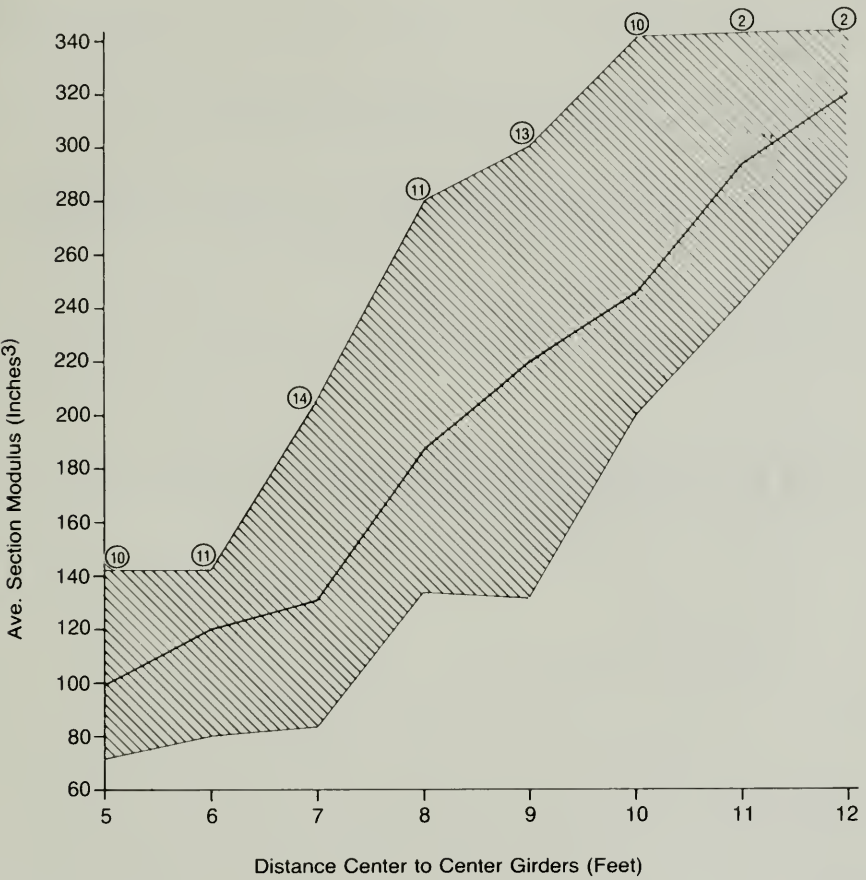

Fig. 4 — Section Modulus of Bridge Ties for Open Decks

4. Eleven of the fourteen did not notch or hole their ties for seating purposes on girder flanges with rivet or bolts heads and remaining three did notch their ties.

D. Tie Hold-Down Anchors

1. Eleven out of the fourteen railroads who responded used hook bolts with plate washers. Three of them also used spring washers. Others used ordinary bolts with washers and lock nuts for security.

2. The spacing of hold-down anchors varied from every tie to every fourth tie. The majority used hold-down anchors on every tie for curved track and on every third tie for tangent track. The criteria was based on the curvature and speed of track.

E. Bridge Tie Spacers

1. Eight of the fourteen respondents stated they used 4″ x 8″, 5″ x 8″ or 6″ x 8″ longitudinal wood as tie spacers. Another four of them used 3″ x 5/8″ thick steel bars as spacers. One of them used both

wood blocks as well as steel bars. One used 6″ x 4″ x 5/8″ angles on top flanges of welded deck girders.

About fifty percent of the respondents still used spacer blocks. Most of them fastened their longitudinal spaces to every tie by means of lag screws, boat spikes or bolts.

F. Guard Rails

1. Eleven out of the fourteen respondents used guard rails and three did not use guard rails on bridges. Six of those who used, stated they did so on designated bridges. In most instances the size of guard rails was lighter than that of the running rails.

2. The clear distance between the heads of running rails and the guard rail varied from 10″ to 11″. However one railroad reported it to be 17″ clear.

3. Six of the eleven using guard rails stated the guard rails extended 60 ft. past the ends of the bridges, two said 30 ft., one 45 ft., one 8 ft. but changing now to 60 ft. and one said 75 ft.

G. Bridge Tie Stresses

1. Currently ten out of the fourteen respondents designed the steel bridge timber ties for Cooper's E-80 loading, one for E-72 and one for E-65. The remaining two did not formally design their ties. Before, eight of them used to design for E-72, one for E-60 and another for E-80.

2. The allowable stresses used for tie design without impact were as follows:
   a) Flexure:
      Twelve of the fourteen: Avg. = 1399 psi, ranging from 1175 to 1650 psi.
   b) Longitudinal Shear:
      Eleven of the fourteen, Avg. = 108 psi, ranging from 75 to 170 psi.
   c) Compression perpendicular to grain:
      Ten of fourteen, Avg. = 477 psi, ranging from 300 to 1000 psi.

The stresses varied depending on the species and grades of timber chosen.

3. Twelve out of the fourteen respondents assumed the axle load to be taken by three consecutive ties for design purposes, while one used the elastic beam method for the design of ties. The remaining one did not respond.

4. Ten of the fourteen stated they did not take any allowance for impact in the design. Only one stated using impact. The figures given in paragraph 2 above, were adjusted, for the respondent using impact.

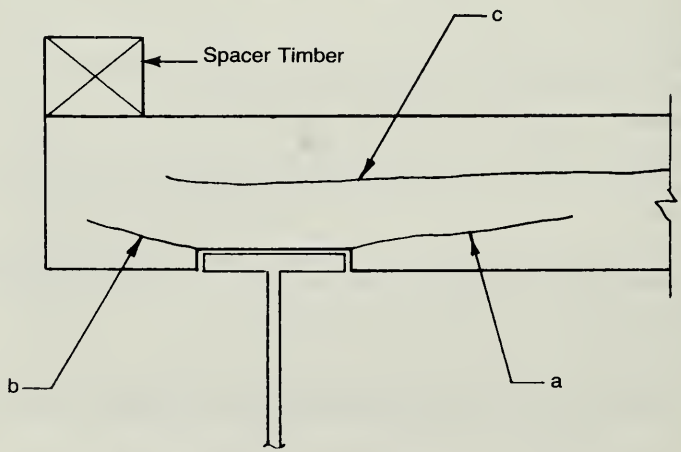

Fig. 5 — Possible Crack Locations in Timber Bridge Ties

H. PROBLEMS

1. Splitting of Ties

Six of the fourteen respondents reported splits between the girders [Type (a)]. See Figure 5. They all used dapped fir and or pine ties. The splitting started at the upper inside corner of the daps and propagated towards the centre of the track or propagated outside towards the ends of the ties [Type (b)]. See Figure 5. This splitting was attributed to daps or knots acting as stress raisers together with steeper grain slopes and/or to wood without free heart. Sometimes this condition may have developed in ties due to service or heavy tonnage. Some respondents stated they stopped using the dapped ties due to the foregoing reasons.

Four of the fourteen railroads also reported splits in ties above the girders [Type (c)]. See Figure 5.

Six of the fourteen who did not experience such splitting problems in ties, a majority of them used only oak ties. An examination of the survey data indicated that the tie splitting did not appear to relate to the stresses used.

2. Skewing or Bunching of Ties

Eleven of the fourteen respondents stated that they had experienced bunching of ties on bridges. The reasons commonly cited were the improper anchorage of rails at bridge approaches, the missing hardware or the damaged guard timber, the inadequate maintenance of hold-down fasteners, the braking of traffic ongrade, and the poor track surface and gauge, etc.

3. Other Problems With Open Decks

Various problems cited with the use of open decks were:

a) Quality of timber
b) Trapped water under ties resulting in corrosion of top flanges of stringers.
c) Poor approaches causing rail level problems on bridge.
d) Plate cutting of ties (partially reduced by use of the tie pads).
e) Susceptibility to fire damage
f) Gauge maintenance problems on curved track in heavy tonnage line.

A few respondents indicated they preferred ballast decks because of the above and because of the fact that ballast decks allowed surfacing on bridges.

CONCLUSIONS

The survey provided a wide range of information on bridge decks of steel bridges which would be useful to railroads.

The examination of the information received did not provide any conclusive indication that the choice of material or the choice of allowable stresses had an effect on the horizontal splitting of bridge ties. However from the responses it appeared that the splitting emanated either above the support points or at the corners of daps may probably be attributed to poor quality of timber, steeper slopes of grains, daps as stress raisers and/or a combination of some other factors.

The loss of tie capacity in longitudinal shear due to overdapping is well known to railroaders as shown by Figure 6.

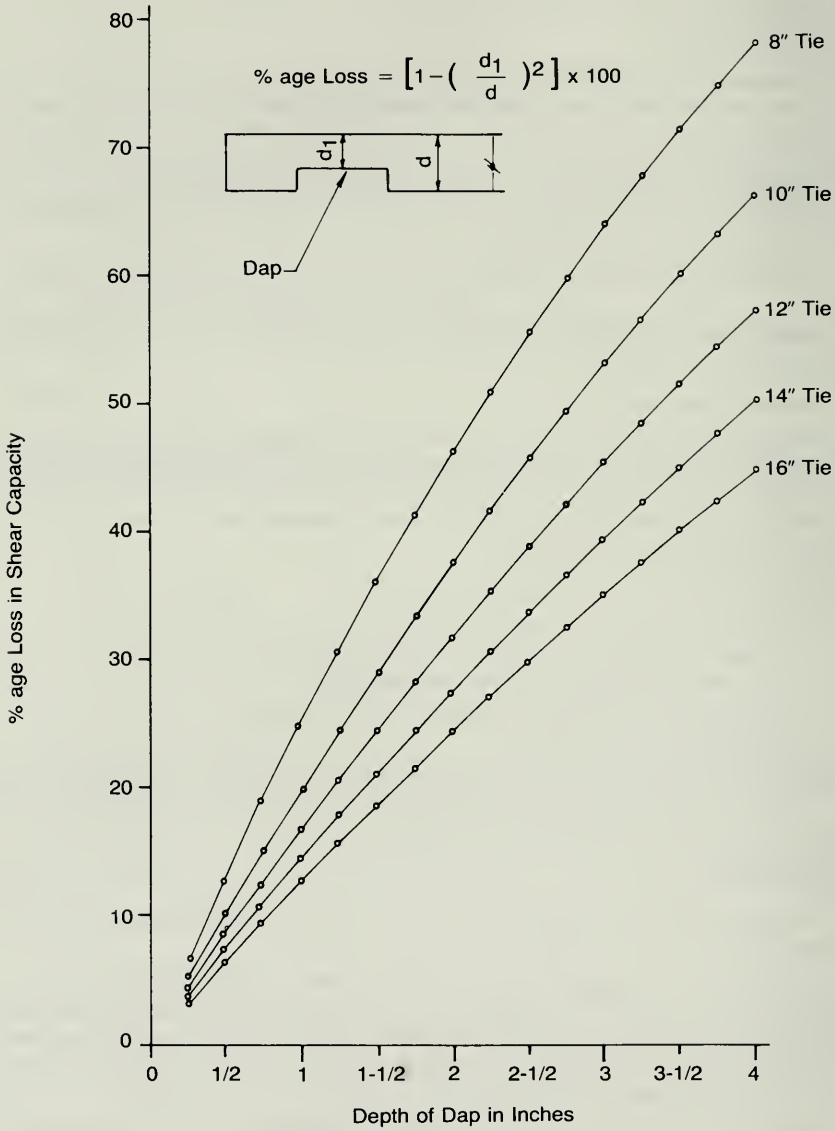

Fig. 6 — Loss of Shear Strength Due to Dapping

ACKNOWLEDGEMENT

The subcommittee D-3-82 of the AREA Committee 7 thanks all the respondents for their time and effort and for the promptness with which they responded to the questionnaire.

COMMITTEE 16—ECONOMICS OF PLANT, EQUIPMENT & OPERATIONS

Chairman: C. Bach

ECONOMIC CONSIDERATIONS IN THE OPERATION AND LOCATION OF AUTOMATIC TRAIN INSPECTION EQUIPMENT

Subcommittee Chairman: R. W. McKnight

Automatic train inspection stations are increasingly cost/beneficial because technology has made it possible for wayside detectors to be coupled with microprocessors and communications links such as base radio stations to report inspection results on a real time basis to train crews, dispatchers and maintenance personnel. Additionally, these automatic inspection stations often prevent catastrophic derailments by promptly alerting train crews of potential hazards.

An automatic inspection station may consist of one or more of the following devices:

— Hot bearing detector
— Flat wheel or wheel impact detector
— Hot wheel detector
— Loose or cracked wheel detector
— High, wide load detector
— Dragging equipment detector.

It is not uncommon for railroads to install dragging equipment and hot bearing detectors at the same location. These detectors are often located 20-25 miles apart along main lines.

High-wide load detectors are often located at yard entrances to check trains for any cars exceeding clearance restrictions that might cause problems.

Before automatic detectors

Prior to the advent of the automatic devices, visual inspection of trains was performed by carmen at terminals and by various employees of moving trains between terminals, including the following:

— Both sides by train and engine crews of their own train on curves.
— One side of each train by crews of trains operating in opposite directions in multiple track territory.
— Both sides of one train at meeting or passing points in single track territory by the crew of the train being met or passed assuming that train was required to stop.
— Both sides by roadway maintenance gangs having two or more employees.
— One side by individual roadway maintenance employees such as signal maintainers and track inspectors operating alone.
— One side at stations, open train order offices and interlocking towers manned by a single employee.

Thus, the frequency and distance intervals of enroute visual inspections are subject to wide variation from day to day in the same territory and the opportunity for such observations is greater during the hours that roadway maintenance forces are actively working. In addition, the efficiency of visual inspection is related to train speed, the experience and diligence of the observer and to weather conditions. Potential journal defects on equipment having roller bearings are more difficult to determine by visual inspection of a passing train.

The use of automatic train inspection devices assures that both sides of each train are monitored with optimum efficiency at selected distance intervals under all weather conditions. These devices when properly spaced, give advance warning of impending failure before it occurs and thereby prevent costly derailments. Nevertheless, though the automatic devices perform the primary inspection function, visual inspection should be continued to the extent available.

Economic benefits

The resulting economic benefits thus include the avoidance of:

— Loss of revenue due to service disruptions
— Costs to repair track and equipment
— Claims for loss of lading and damage to adjoining private property.
— The cost of clearing derailments resulting from undetected equipment failures.
— Added freight car hire for all per diem cars delayed due to disruption of service.
— Added non-productive crew costs for crews adversely affected by the disruption of service.
— Detour expense, if involved
— Potential personal injuries to employees and outsiders.
— By reducing accidents, a reduction in the public and governmental perception that railroads constitute a public menace which should be closely regulated. (For example, speed restrictions could be imposed on freight trains during rush hours on tracks adjacent to highways, computer or transit tracks.) Also, reduced insurance costs could result from fewer accidents.

Each of these cost categories would be magnified in degree should the damaged cars contain toxic and/or highly flammable lading. While they are not subject to precise quantification, they are potentially substantial and could significantly impact net income. Likewise, the adverse effect of derailments on customer relations is not measurable but is a real factor for consideration. With the continuing advent of intermodal and double stack business, which is a service oriented and competitive market, the railroad industry cannot afford service disruptions. Additionally, many contracts are now written with bonuses or penalties for performance, which could be affected by derailments.

Other savings from the use of automatic inspection stations include:

— Reduction in personnel to inspect rolling stock.
— Reductions in clerical costs to prepare and/or handle inspection reports by automatic reporting to train crews and dispatchers via radio.
— Preparation of written reports automatically by microprocessors.
— Reduction in record keeping and elimination of paper chart recorders, paper supplies and maintenance of the recording devices through data transmission and computer usage.
— Reduction in maintenance costs due to the ability to remotely test and check on the condition of the wayside detectors from a central location via microprocessors and self-diagnostic equipment. (Some hot bearing detectors do a diagnostic self-check periodically.)

Although it is not possible to accurately predict savings, equipping detector locations with "talker" or voice radio to alert train crews has resulted in their being notified promptly as soon as the train clears the detector site and avoiding further train delay in cases where no defect is found, such notification is made to the train crew via the "talker" so they can proceed.

One major railroad installs hot bearing and dragging equipment detectors at the same location with a microprocessor to handle and analyze the data from the detectors. Although it is a stand alone system (the alarm decision is made at trackside) and the results are sent via digitized voice radio to train crews, the entire data results are sent to the dispatcher's office and displayed on a video display unit or CRT. Data from all detectors on a division are sent to the division headquarters where the information is stored and analyzed by a larger computer so maintenance and diagnostic practices can be reviewed or revised. Other data automatically generated includes number of axles in train, length of train, date and time of train passing inspection station, speed and direction.

Automatic train inspection stations are a practical tool for safer and more efficient railroad operations.

COMMITTEE 22—ECONOMICS OF RAILWAY CONSTRUCTION AND MAINTENANCE

Chairman: W. C. Thompson

ECONOMICS OF RAIL SERVICE LIFE AREA STANDARD VS. PREMIUM RAIL

Subcommittee Chairman: J. M. Johnson

EXECUTIVE SUMMARY

This report examines the use of so-called "premium rail" by a number of North American railroads, both large and small. It is primarily based on questionnaire responses solicited in the study. The study does *not* include comparisons of lubrication, rail fastenings, track modulus, tie type or condition, or continuous welded rail as factors in rail life considerations. In other words, these factors are considered to be "equal."

The study found that nearly all large railroads have a formal policy on the use of premium rail on curves, ranging from standards based solely on curvature to others who have developed computerized economic models considering rail wear rates, defect rates, price differentials, etc. All recognize that premium rail gives an improved service life, with estimates of service life improvements ranging from 25 to 300 per cent. This improvement can be translated into savings in capital investment, gang operations, equipment, inventory and track occupancy costs. There was some disagreement as to savings in welding and rail grinding.

Although lubrication was not considered, it should be noted that lubrication clearly improves rail service life and should be considered *before* considering premium rail.

Railroads currently install premium rail on curves, but many engineering managers recognize the distinct possibility that the onset of 125 ton cars or a reduction of the price differential between standard and premium rail will likely result in the installation of premium rail on tangent track as well.

INTRODUCTION

This study examines the economics of the use of premium rail on curves. "Premium rail" as opposed to AREA standard rail is rail with a Brinell Hardness Number (BHN) greater than 340. There are a variety of heat treated and alloy rail available with this hardness. Naturally, there is a cost premium for the extra treatment. This study assumes a margin of 25 percent above the cost of standard rail, but each user must develop his own price factor based on the current market and his relationship with the supplier.

The study considers effect of curvature, tonnage, wheel loads, grades, and traffic environment. However, the following are treated as being essentially "equal" in each problem situation: rail lubrication, jointed vs. welded rail, track fastener design, rail wear replacement limits, track modulus, tie condition, and general track condition. Improvements in these areas usually have economic benefits and engineering impact far beyond the service life of the rail involved.

The committee's questionnaire was distributed throughout North American railroads in the fall of 1988 and responses were received from most large railroads and a few of the smaller ones. The rail use policies presented, based primarily on curvature and tonnage, are discussed and illustrated. Other factors are discussed throughout the report and are the basis for discussion of the various economic factors.

Finally, a recommended method or standard for the use of premium rail is presented. Additional variables should be considered by the user, depending on the particular situation.

FACTORS TO CONSIDER WITH PREMIUM RAIL

Curvature

At this time curvature is the primary consideration in the use of premium rail on most railroads. In fact several use this as the *only* consideration. Responses to our questionnaire indicated that curves in

excess of two degrees represent as much as 15 percent of a given railroad total mileage (see Table 1). This represents a substantial investment and is particularly expensive to deal with when curve rail must be transposed or replaced on a spot basis inconsistent with the overall service life of the rail in that track segment. Railroads typically use smaller, more expensive "curve relay" gangs to perform this work. Several railroads also reported that premium rail use in the total overall relay program was 10 to 20 percent of the total, indicating that shorter curve rail service life was impacting main line relay requirements as well.

Again, it should be pointed out that rail lubrication should be considered *before* the use of premium rail. There are many curves and traffic environments where lubrication will increase the wear life of standard rail such that it is more economical than non-lubricated premium rail. Also, premium rail should be lubricated in most environments. FAST (Facility for Accelerated Service Testing at Pueblo, CO) results indicated service life extensions of up to 500 percent with lubrication. Some railroad service tests of lubrication are not as dramatic, but still indicate improvements of 150 to 200 percent. In many cases, this will bring the curve rail service life to that of the adjacent tangent track where accumulated fatigue defects will likely dictate economic service life. Lubrication is also discussed in terms of sanding problems under "Grades."

Table 1

Curve Distribution of Various Railroads
Curves as a Percentage of Route Miles

RR	RR Size (Miles)	Degree of Curvature 0-2	2-4	4-6	6-8	8+
A	>10,000			5 →		
B	<10,000	3	1			
C	>10,000		15 →			
D	<10,000	4	2			
E	>10,000	14	5	1		
F	>10,000	8	4	1		

Several railroads make premium rail decisions based solely on curvature, while others include annual tonnage in the standard. Table 2 shows some of these standards and they are shown graphically in Figures 1 and 2.

Table 2

Curve Rail Replacement Standards
For Various Railroads

Railroad	Premium Rail Standard
	(Figure 1)
A	All Curves Over 1 Degree
B	All Curves Over 1.5 Degrees and Over 10 MGT
C	All Curves Over 2 Degrees
D	All Curves Over 2 Degrees and Over 10 MGT
E	All Curves Over 2 Degrees and Over 25 MGT
F	All Curves Over 4 Degrees
G	All Curves Over 4 Degrees and Over 25 MGT
	(Figure 2)
A	Curvature/Tonnage Combination
B	Curvature/Tonnage Combination
C	Curvature/Tonnage From Computer Model

Annual Tonnage

Both Canadian and American railroads are presently rationalizing their route structures by abandoning or selling low-tonnage lines and eliminating redundant routes. Traffic is concentrated on "core routes" and freight scheduling demands are often as time-sensitive as passenger service once was. Higher tonnage and unit train operations reduce the ability of rail traffic to work-harden new rail before detrimental gage face wear begins, resulting in shortened service life. Track occupancy also becomes more expensive.

Although not all railroads consider tonnage in premium rail decisions, those that do (typically the larger ones) use tonnage in relation to curvature (see Figures 1 and 2). Obviously, premium rail will be more economically attractive in situations where curvature and tonnage combine to shorten service life.

This is apparently an area where individual railroads should look carefully at the economics of premium rail use in two areas. First, the "blanket curvature-based standards may prompt the use of premium rail in low tonnage situations where service life may be much longer than is needed. Second, low-degree curves on high tonnage lines (greater than 50-60 MGT) may offer attractive economic return with the use of premium rail.

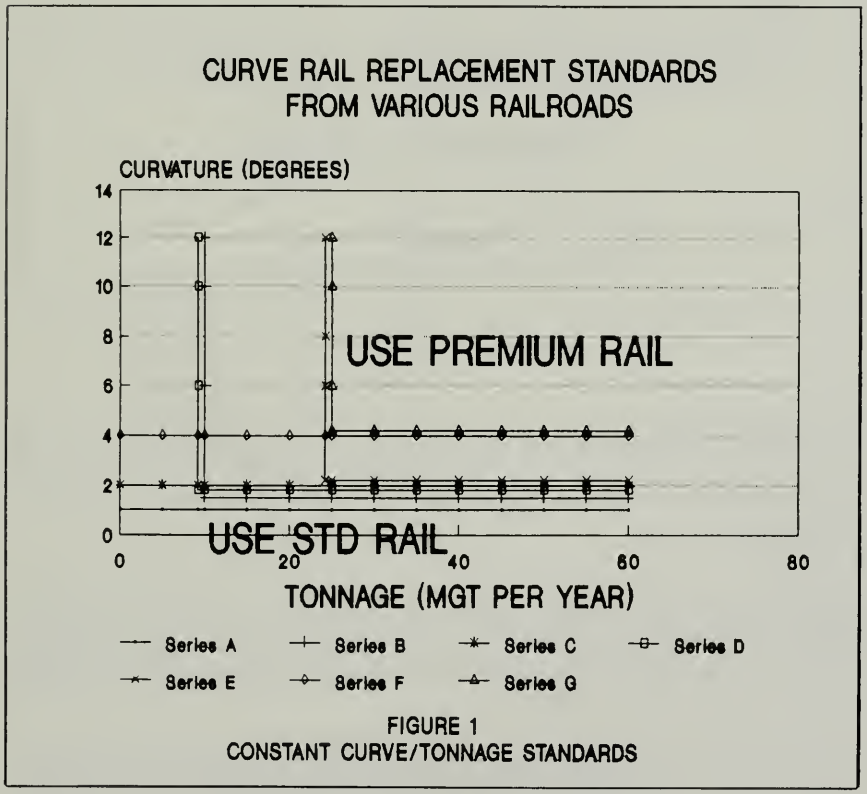

FIGURE 1
CONSTANT CURVE/TONNAGE STANDARDS

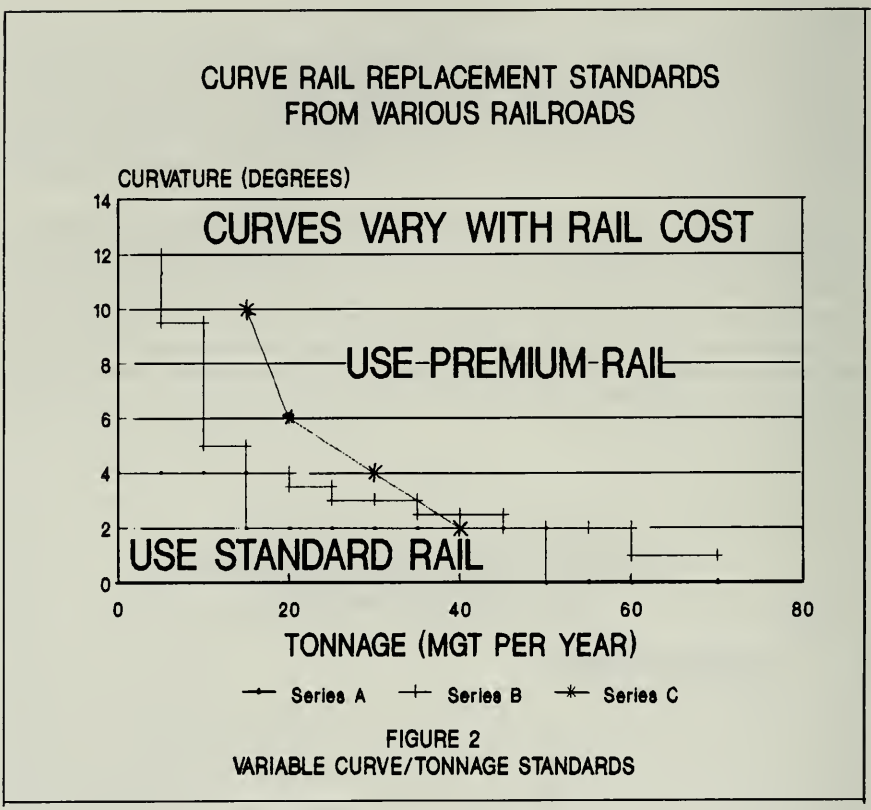

FIGURE 2
VARIABLE CURVE/TONNAGE STANDARDS

Wheel Load Distribution

Our responses showed that wheel loadings (i.e. unit train) are not directly considered in premium rail standards by most railroads. However, unit train operations will aggravate curve rail wear rates and should be included in the analysis. For example, one railroad uses premium rail where the proportion of heavy wheel loads exceeds 40 percent. Another adjusts the curve/tonnage standard downward where unit trains are involved.

Traffic Mix

No formal standard was reported in this area, but several indicated that the field manager has latitude to consider traffic mix in premium rail decisions. This is an area for additional study, particularly as the industry develops and uses most articulated intermodal vehicles. Unit trains of 89-foot flat cars and double-stack articulated container trains present particular curve wear problems and these problems should be considered.

Rail Grinding

There was no indication of the use of rail grinding planning with respect to premium rail installation standards. There is some disagreement as to whether premium rail reduces or increases rail grinding costs.

Track Profile

Track profile is again an area where the field manager may consider special premium rail installations. Premium rail may have a good economic return on heavy grades where there is heavy locomotive sanding since lubrication is difficult to control and may not even be desirable.

Since curvature and steep grades frequently occur in the same location, severe curvature/grade locations on tonnage routes (greater than 20 MGT) yield excellent economic return with curve rail service life extensions of 200 to 300 percent with the use of premium rail.

MODELING AND FIELD INSPECTION

The AAR has developed an economic analysis model based on rail defect rates and is also developing a model that evaluates curve rail wear rates. Additionally, a few of the larger railroads have their own economic models. All these models typically allow discounted or present value comparisons of different rail installation options by analyzing various variables such as curvature, tonnage, traffic type, gang costs, material cost, capital cost, etc. All railroads report using a variety of manual and automated field inspection methods to establish or verify the need for premium rail at a given location.

PREMIUM RAIL TYPES

Questionnaire responses indicated that the use of head-hardened (HH) and fully heat treated (FHT) premium rail was much more common than the use of various alloy rails. The alloy types include Chromium, Chrome-Molybdenum, and Vanadium. Talbe 3 outlines some of the responses.

Table 3

Premium Rail Use Data for Various Railroads
5-Year Use by Rail Types

RR	RR Size (Miles)	Premium Rail Type					
		HH	FHT	Chrome	Ch-Moly	Vanad	Other
A	>10,000	3	94			3	
B	>10,000		100				
C	>10,000	73	27				
D	>10,000	9	2	3	85	1	
E	>10,000	90	10				
F	<10,000	100					
G	>10,000	89	10	1			
H	<10,000	14	2		80		18

ECONOMICS IMPROVEMENTS WITH PREMIUM RAIL USE

By increasing the service life of curved rail, premium rail will reduce costs in several areas:

Capital Rail Investment

With rail life extended, the need for annual curve rail renewal is reduced. The total savings for capital investment was given as $275,000 per mile by one railroad. This amount would include material and installation. Another indicated a 15 percent reduction on total curve rail costs.

Gang Costs

Curve rail replacements are typically much more expensive than main line relays on a unit or per foot basis. This is caused by higher mobilization, transportation, and material handling costs. Therefore, it is attractive to bring all portions of a line segment—tangent portions and curved portions—to the same service life by using premium rail. Furthermore, reductions in curve rail replacement can ultimately lead to staff reductions.

Equipment

The following equipment needs may be reduced by extending curve rail life:

1) gang equipment—truck, cranes, spikers, etc.
2) rail welding equipment—both plant and field machines
3) rail transportation equipment—loading and unloading

Track Occupancy

With tonnage being concentrated on "core routes," delay costs to both gangs and trains can be extreme. Any reduction in track occupancy requirements by track gangs translates directly into improved train movements and a more consistent business plan. This will continue to grow as an economic factor as operating crew districts become longer and longer and as traffic is concentrated more and more. Even minor delays to trains will cause major operating and economic problems.

Rail Inventory

While not all respondents agreed, overall inventory costs should be reduced as rail service life is extended by the use of premium rail.

Rail Welding

Again, there is disagreement, but overall plant and field welding costs should go down with reduced rail requirements. However, there are some problems associated with welding premium rail. In particular, there are special concerns when making thermite welds on premium rail.

Grinding

There was also disagreement as to whether grinding costs go up or down with premium rail. It may in fact be more difficult to grind premium rail since it is harder, but the frequency of grinding and the number of grinding and the number of grinding passes should be reduced.

Comments

Each of these factors should be considered on an individual railroad basis with careful analysis of requirements and costs. This is particularly true with larger systems. Smaller low-tonnage routes may not need such detailed work. In fact one respondent from a small railroad said the "premium rail policy" for his railroad was the application of work-hardened relay rail on curves from tangent areas.

125 TON CARS

The possibility of heavier axle loads is of great economic concern to those responsible for rail planning. Some studies project that rail service life will be cut in half or even more by the introduction of heavier cars. Since rail costs are so high, the possibility of premium rail use in tangent areas will have to be considered. In the tangent environment, the fatigue life of premium rail will be an issue.

CONCLUSIONS AND RECOMMENDATIONS

Our study found a variety of policies regarding the use of premium rail. Most large railroads have made substantial savings by carefully outlining curvature/tonnage policies for premium rail use. Others have only applied a curvature standard. In both cases the price differential between standard and premium rail is considered. Most railroads indicate a substantial improvement in rail service life, ranging from 25 to 300 percent under various conditions. This translates into savings in a variety of cost factors.

The use of premium rail where curvature exceeds two degrees is a common standard. Railroads not already considering tonnage in their analysis should contemplate raising this limit on low tonnage routes (below 15 MGT) and lowering the limit to one degree on high tonnage routes (above 50 MGT).

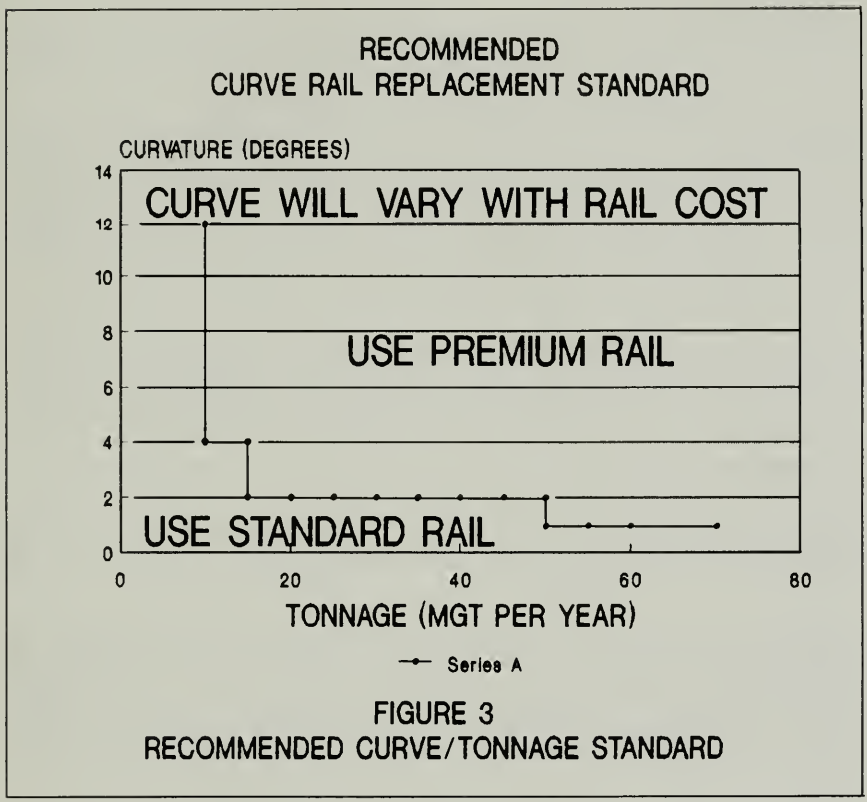

FIGURE 3
RECOMMENDED CURVE/TONNAGE STANDARD

Figure 3 outlines this recommended standard. This standard should be examined periodically as the price differential between standard and premium rail fluctuates. Also, individual line segment or railroad characteristics such as unit train traffic, heavy grades, etc. should be considered and the standard modified appropriately.

Finally we wish to thank all those who assisted with the preparation of this report, especially those who furnished information.

THE SAFETY OF RAILWAY BRIDGES

By: Louis T. Cerny*

As Executive Director of the AREA, one question that comes up frequently on my job in Washington is why there are no Federal railroad bridge safety standards as there are Federal track safety standards. Perhaps the primary reason is because the railroads' record is so impeccable on this subject.

It has been over 50 years since there was a fatality due to a structural collapse of a railroad bridge on any United States railroad and this must stand as one of the most exemplary transportation safety records in history. After the 1967 collapse of a highway bridge across the Ohio River, at Point Pleasant, West Virginia which killed 46 people, federal investigations into the subject lead to recommendations that highway departments use bridge inspection policies similar to that used by railways. Railway bridges are generally inspected at a minimum of once a year with more frequent inspections taking place in situations where they have been judged by experienced engineers to be necessary.

Railroads companies realize that the bridges are perhaps the most critical individual elements in the continuity of a railroad line and they have every incentive to protect their investment and prevent bridge collapses from occurring. The Federal Railroad Administration has authority to regulate railroad bridge safety, but detailed investigation into his matter has lead to the conclusion that federal railroad bridge safety standards would not improve safety, and in fact could be counter productive, by taking personnel from the limited pool of experienced railroad bridge engineers to serve in administering the bridge safety inspection program, leading to a situation when less talent was actually available for the inspection of the bridges themselves.

There have been many instances where congressional or citizens complaints about railroad bridges conditions have been referred to the FRA and these investigations have revealed the safety of the bridges despite cosmetic deterioration.

The American Railway Engineering Association has 24 technical committees of which 3 deal with bridges. They are: Committee 7—Timber Structures; Committee 8—Concrete Structures and Foundations; and Committee 15—Steel Structures. Each of these three committees is responsible for a chapter in the AREA Manual of Recommended Practice. These three chapters have the same numbers as the committees. As is true in all the work of the AREA, the material in these chapters is not meant to represent minimum safety standards and takes into consideration the need, in both design and maintenance procedures, to achieve an economical long life for the structure.

This is an era of many railroads mergers and the increasing efficiency of railways. The increase in rail efficiency can be shown dramatically by the fact that last year was the all-time record in the United States for railroad ton-miles, about one-third higher than the legendary days of World War II. This was accomplished with perhaps 1/8 as many people as were employed during that time and with a much smaller amount of trackage. This efficiency has been brought by new technology, new methods of track maintenance, new methods of train dispatching and new technology in locomotives, cars, signals, and track materials.

Because of this increased efficiency, the need for many railroad structures or the use of their complete capacity in terms of numbers of tracks may no longer be necessary. For the particular bridges affected, economy of maintenance into the indefinite future may not be a prudent economic consideration. Also, as occurred in my own personal experience on one of the newly created short-line railroad companies, the funds simply may not initially exist to maintain the structures to long term economical maintenance standards. The question then is to maintain the bridge safely for operation, because if the new short line railroad company does not survive its first years of operation, the question of long term maintenance economy is academic.

Therefore, even though I am Executive Director of the AREA, I can point to occasions in my career where the correct engineering decision was to maintain bridges in such fashion that they did not comply with AREA recommended practices.

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*Executive Director, American Railway Engineering Association. Adapted from his address to the Sixth International Bridge Conference, Pittsburgh, PA, June 12, 1989.

One of the reasons railroads have such an excellent bridge safety record is due to the historic conservativeness of railroad design and I would like to take you through a hypothetical historical situation to indicate one of the reasons why this would be so.

Let us take a civil engineer graduating over a century ago in 1880, who then went to work for a bridge department of a railroad. In 1880 the heaviest things crossing the bridges would be locomotives that we might associate with old-time movies of the "wild west," very small steam locomotives of the 4-4-0 or 2-6-0 type. Now let us say that this civil engineer advances in his career with the railroad and 40 years later he is chief bridge designer for the railroad. By this time in the early 20th century the locomotives have grown to monstrous proportions judged by the standards of the 1880's, with locomotives weighing six times and more the weight of those in 1880. Picture him sitting behind his desk in 1920 as he makes the final decision on a bridge design, trying to imagine what railroading would be like 40 years from then.

With the extraordinary changes in size that he had seen in locomotive design from 1880 to 1920, we could imagine the thoughts going through his head of what the locomotives of the present time would look like, and his thinking that he had better really design for locomotives that would be as heavy in relation to 1920 designs as those of 1920 were to those of 1880.

What this chief bridge designer could not foresee was the advent of the diesel locomotive which would allow high horse power to be spread over a multiplicity of individual locomotive units. Thus the bridges were in many cases vastly over-designed in relation to the load they actually now carry. This is especially true when it is compounded by the fact that bridges then were designed for the"hammer-blow" caused by the reciprocating parts on the side of the steam locomotives, which by AREA recommended practices was an addition of 100% over the static load.

The largest steam locomotives had axle loads much higher than those in normal use on today's locomotive or freight cars. The heaviest normal freight cars operating in US practice have axle loads of 66,000 lb. while some steam locomotives had axle loads over 80,000 lb. which, when considered with the hammer blow effect, was the equivalent of a 160,000 lb. static axle load. Another feature of railroad bridge design was the use of riveting or bolting instead of welding. This minimizes the danger due to propagation of fatigue cracks.

I would like to relate a personal experience, because there have been several recent incidents in which engineers have made statements that have alarmed the public about railway bridge safety and caused much unnecessary use of railway bridge engineering talent in handling questions of reporters and other people not experienced in the discipline of railway bridge engineering. One of these situations involved the Northeast Corridor in the early part of this decade in which a report used words such as critical and substandard to describe the condition of bridges which were in a completely safe condition.

Having inspected bridges for many years myself I asked which was the very worst bridge, and was told the Woonasquatucket River Bridge just north of the station in Providence, R.I. So I made arrangements to inspect it myself accompanied by railroad bridge inspectors and others. This ballasted deck, deck truss span, had the most extensive deterioration I had ever seen on a steel railroad bridge. The deterioration was so advanced that some of the secondary members had rusted completely through and whole sections had fallen out. On some main members an entire flange had completely corroded and was missing. The railroad bridge inspector assured me that the bridge was safe and indeed calculations confirmed the judgement of the railroad inspector in a very emphatic way. The calculations came out that the bridge, *in that condition,* could carry *17 times* the live load to which it was actually subjected without stressing the material beyond the allowable limits. While obviously such deteriorated conditions would not be acceptable in some circumstances, this inspector had correctly determined that in this particular case, extensive deterioration could be tolerated without compromising safety.

Similar situations which have caused railroad bridge engineers to needlessly spend time reacting to complaints are situations where bridges were built to carry double track, but the second track was never added. In many of these cases extreme deterioration of the columns, especially where the corrosion is over a short distance, can safely be allowed to occur. This can occur at the bottom of columns at streets where salt action may occur, and can exceed 50% of the column cross section without even approaching any type of safety limit.

I would suggest a mental exercise which is occasionally useful for bridge engineers or other design engineers to go through to keep a perspective on their work. This is for bridge engineers to imagine that their job is to actually predict the load which would collapse a bridge. In other words, load will keep being added to the bridge and the goal is to predict as accurately as possible what load will result in the bridge's actual collapse. This would then involve no factor of safety or any of the conservative assumptions which are normally made in design. The conservative assumptions combined with the factors of safety give an actual ratio between the permitted load and the actual collapse load many times what most engineers realize. In one such exercise I undertook myself I was rather amazed to find that I could not depend on collapse even by cutting the stringers all the way through, as the stringer's strength as cantilevers actually could take the load and would prevent the bridge from collapsing.

In the vast majority of cases railway bridges have been designed with such a conservative philosophy that the ratio of the actual collapse load compared to the design load is normally much higher than in highway bridge design. Railway bridge design has tended to emphasize a high degree of redundancy and avoidance of the type of detail that was involved in the Mianus River Highway Bridge collapse in Connecticut. Multiple load paths which would permit the bridge to remain intact despite the loss of an entire member are more frequent in railway design than in highway design.

In the U.S. there are about 130,000 railway bridges with a total length of about 3,100 miles. Recent information indicates that about 43% of these bridges, representing about 33% of the total length, are timber bridges. While those who have worked extensively with timber recognize the virtues of timber as a modern engineering material, and probably have some rather incredible stories about the ability of timber to take impact loads, there is often a perception among the public that there is something deficient or at least obsolete when timber is used. Modern retreatment techniques, where the timber bridge can be retreated in place, have in some cases made the life spans of timber bridges comparable with those of steel and concrete. I am sure that those of you familiar with timber have heard the stories about when timber is preserved or not exposed to air, strength can last essentially indefinitely. This is evidenced by untreated timber pile foundations that still support stone bridges carrying traffic, even though the bridges were built by the Romans more than one thousand, seven hundred years ago.

In summary, it is our duty as railway engineering professionals to keep safety first in mind in maintenance of railroad bridges-safety of the general public, the railroad's passengers, and railroad employees. History shows what an excellent job we have been doing, and it is through continuing high quality of inspection that our fatality-free record will continue.

MEMOIR
William J. Hedley

On Sunday, September 10, 1989, former AREA President William J. Hedley, 86, died of infirmities at his home in Clayton, Missouri, the town which twice elected him mayor.

Bill Hedley joined the AREA in 1931 and become a director in 1950. He was elected junior vice president in 1954 and began his climb to the presidency which occurred in 1956. In 1965, Bill was elected to Honorary Membership in the AREA. This honor is bestowed to those of acknowledged eminence in railway engineering and is limited to ten living persons. Bill's committee activity began in 1938 with Committees 9 and 14 with chairmanships of both. In 1969, Committee 9 elected him Member Emeritus as did Committee 14 in 1982. This class of committee membership was established to permit recognition of long-sustained meritorious service of committee members following their retirement, and Bill Hedley certainly fit this description.

A review of Bill's AREA membership file revealed a world of activities, accolades and recognition that few of us are aware of. For instance, in 1973 Mr. Hedley was the recipient of the Hoover Medal founded in 1929 to commemorate the civic and humanitarian achievements of Herbert Hoover. Others who have received this prestigious medal are Karl T. Compton, Alfred P. Sloan, Jr., Charles F. Kettering, and Dwight D. Eisenhower. Bill was recognized because he was a "distinguished civil engineer, leader of his profession, and public servant whose accomplishments demonstrate how professional engineers can serve the public and protect the mobility of the nation through improved transportation."

Mr. Hedley graduated in 1925 with a bachelor's degree in civil engineering from Washington University in St. Louis. For a short time he worked as a draftsman for the Mississippi Valley Structural Steel Co. before beginning a 42-year career, also as a draftsman, with the Wabash Railroad which later became a part of the Norfolk & Western.

He was appointed chief engineer at the Wabash in 1957 and retired in 1967. But Bill didn't really retire, he simply changed jobs. Among other things he became a consultant to Sverdup & Parcel and the Department of Transportation, being officially recognized in a special letter of appreciation by then-Secretary of Transportation John A. Volpe which was read at the National Conference on Grade Crossing Safety. He wrote a multitude of nationally distributed articles on railroad-highway grade crossings, which he worked diligently to improve. Almost from the beginning of his railroad career Hedley was assigned to work involving railroad-highway crossings and grade separations. This soon expanded into a variety of projects and activities involving the relationship of railroads to public works and brought about ever-broadening contacts with public agencies at every level.

Mr. Hedley was president of the American Society of Civil Engineering, and the Engineering Society of St. Louis, and chairman of the Interprofessional Council on Environmental Design. Bill was a member of The Railway Tie Association, Roadmasters & Maintenance of Way Association, and the American Railway Bridge & Building Association. He served on the Executive Committee of the Highway Research Board (now Transportation Research Board) from 1968 to 1971. He served on the executive boards of state railroad associations in several states and was a member of four Chambers of Commerce. Washington University bestowed him with an alumni citation in 1966. Hedley's other technical and professional memberships included the International Association for Bridge and Structural Engineering, National Defense Transportation Association, and the Missouri Society of Professional Engineers which elected him St. Louis Chapter's Engineer of the Year in 1964.

Bill is survived by his son Dr. William Henby Hedley of Dayton, Ohio, and his brother, J. Henry Hedley of Coffeyville, Kansas.

J. A. Meyerhoeffer

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